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SAN FRANCISCO MASTER PLAN FOR WASTE WATER MANAGEMENT

PRELIMINARY COMPREHENSIVE REPORT

PREPARED BY
DEPARTMENT OF PUBLIC WORKS
SEPT. 15, 1971

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SEPTEMBER 1971

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CITY AND COUNTY OF SAN FRANCISCO

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PREFACE TO 2ND PRINTING

This represents the 2nd printing of the three books comprising the Master Plan for Waste Water Management, 1971. Several corrections to the text and plates have been incorporated into this edition as well as major additions to the Appendix including a comparison of San Francisco's Master Plan to the 1969 Bay-Delta Regional Plan, and a description of the design characteristics of the proposed Lake Merced Water Pollution Control Plant.

Persons and agencies desiring the corrections and additions may obtain them from the Division of Sanitary Engineering on request.

First editions were distributed to the following agencies, firms and individuals:

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2. U.S. Army Corps of Engineers

State Government Agencies:

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2. Dept. of Fish & Game
3. Dept. of Public Health
4. Regional Water Quality Control Board (2 copies)

Local Government Agencies:

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PREFACE

This Master Plan is one of the sub-regional studies now in progress throughout the Bay area from which a regional water quality management plan can be evolved. The Plan consists of a Summary Report prepared for non-technical decision makers, and a Comprehensive Report with its text and book of plates bound separately for the reader's convenience.

This Master Plan also is constituted by the technical reports on the List of References and an organized file of work papers to substantiate the findings and recommendations.

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17	Dissolved Air Flotation - Appendix G - Performance Evaluation of Baker Street Facility with Raw Sewage	7/71	(2)
18	Report on Water Pollution Control Plants - Report 1 - Phase I - Existing Operations and Plant Performance	9/71	(3)
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21	Report on Water Pollution Control Plants - Report 2 - Phase II - Alternative Treatment Processes for Reductions of Toxicity and Biostimulants	9/71	(3)
22	Report on Water Pollution Control Plants - Report 3 - Phase I - Reductions Required for Toxicity and Biostimulants	9/71	(3)

SUPPLEMENTARY REPORTS (Cont.)

Ref. No.	Title	Date	Prepared By
23	(Not used)		
24	City & County of San Francisco, A Predesign Report of Marine Waste Disposal, Oceanographic and Base Data Acquisition and Evaluation of Alternate Locations	9/71	(3)
25	Survival of Dungeness Crab Larvae in Two Concentrations of San Francisco Sewage Effluent (This is printed as part of Reference No. 24)	2/70	(8)
26	Interim Water Quality Control Plan, San Francisco Bay Basin	6/71	(9)
27	San Francisco Bay Delta Water Quality Control Program, Final Report	6/69	(10)
28	City and County of San Francisco Sewerage System - Basic Data Development (Land Use and Population)	6/71	(1)

(1) San Francisco Department of Public Works

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(6) Dr. Wheeler J. North

(7) Water Resources Engineers, Incorporated

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(9) California Regional Water Quality Control Board San Francisco Bay Region

(10) Kaiser Engineers

CHAPTER I

INTRODUCTION

To preserve and enhance the waters of San Francisco Bay and the Pacific Ocean and to protect these waters for all the beneficial uses to which these waters are put, at all times -- these are the goals that have caused the San Francisco Bay Regional Water Quality Control Board to request the City and County of San Francisco to prepare and submit a City-wide sewerage Master Plan. This Master Plan is to formulate the facilities, costs, and construction schedules necessary for the control of pollutants going into the Bay and Ocean from the City's combined sewer system.

The history of water pollution control problems and facilities development in the City and County of San Francisco dates from the mid-1800 era when the first efforts were made to build sewers to remove the City's sanitary wastes and street washings from population centers to the nearest water courses for disposal. As was common practice in that period, the house plumbing and sewers were constructed as a combined system to carry both storm runoff and sanitary wastes in a common conduit. Brick sewers were constructed which are still in use in older sections of the City. Catchbasins were installed on almost every corner of the City as was considered necessary during the era of horse-drawn vehicles. Because of San Francisco's compact and early development a totally combined sewerage system has been created which is unique on the West Coast. About 100 large, older cities throughout the nation have such combined systems and several such cities on the West Coast have major areas served by a combined system. To date there have been two master plans developed for the sewerage system of the City and County of San Francisco, one in 1899 as shown in Plate I-1 and the other in 1935. The objective of the two previous master plans was to provide direction to control the City's dry weather pollution problems. This Master Plan of 1971 is directed toward the control of the 6 billion gallon yearly volume which overflows from the City's combined sewer system during the winter period.

In the past 20 years water quality of the Bay around San Francisco has been improved by the efforts of both the dischargers and the San Francisco Bay Regional Water Quality Control Board. With the attainment of significant net reductions in pollutant emissions of dry weather waste flows, and in response to increasing public pressure for further water quality enhancement, attention has increasingly been given to resolving the wet weather pollution problem of combined sewer overflows. In many sections of the United States today sewer overflows represent a significant source of water pollution. This report will define the effect of San Francisco's combined sewer overflows upon the waters of San Francisco Bay and the Pacific Ocean.

The Role of the Regional Water Quality Control Board

The present system of water quality control in California began in 1949 with the passage of the "Dickey Water Pollution Act" which provided the basis for today's comprehensive State water quality control laws. Jurisdiction of the State's waters is vested in the State Water Resources Control Board through the California Water Quality Control Act. The State Board implements state objectives and policy through nine Regional Water Quality Control Boards. Each board consists of nine members appointed by the Governor for a term of four years and is charged with developing water quality control policy and objectives for their respective regions, setting of discharge requirements and the enforcement of these requirements by cease and desist orders, restriction of building permits and monetary penalties for failure to comply with discharge requirements.

The San Francisco Bay Region consists of portions of all nine Bay Area counties and is under the jurisdiction of the California Regional Water Quality Control Board No. 2.

In the development of policy and objectives which are ultimately formulated into discharge requirements, the Regional Board must assess the beneficial uses of the waters under their jurisdiction and then develop individual discharge requirements to protect these beneficial uses or on some waterways involving limited exchange of water masses and resultant assimilative capacity, prohibit all discharges. Requirements when set must be met in order to protect the designated beneficial uses to enhance the quality of the waters of the State.

Beneficial uses of the waters contiguous to the City and County of San Francisco include:

1. Swimming, wading, pleasure boating, marinas, boating, ramps, fishing and shellfishing.
2. Fish, shellfish and wildlife propagation and sustenance, migratory bird habitat and resting.
3. Industrial cooling water.
4. Firefighting and industrial washdown.
5. Navigation channels and port facilities.
6. Esthetic appeal.

Requirements for each of the City's three treatment plants have been set by the Board.

The Role of the Board of Supervisors

To demonstrate good faith the Board of Supervisors in October 1968 passed Resolution 716-68 which stated in part the intention of the City, based on stated cost estimates, to comply with the requirements of the Regional Board for wet weather discharges for those waters westerly of Pier 45 by July 1981 and for those waters easterly of Pier 45 at a date to be determined as the need arises.

The Board of Supervisors has also authorized the expenditure of over $2 million of City funds which has been spent, in conjunction with about $1 million of federal funds, on studies to characterize and investigate the combined sewer overflow problem. Included in this is the Baker St. Dissolved Air Flotation demonstration project.

Other measures aimed at the reduction of water pollution recently taken by the Board of Supervisors include the adoption of a new and strengthened industrial waste ordinance, a part of which is to provide source control for industrial wastes which cannot be removed at the City's treatment plants and the adoption of a sewer service charge to more closely relate future financing to those who use the sewer system.

Existing Sewerage System

The Master Plan alternatives to be evaluated in this report must not only necessarily address the same realities as govern the existing sewer system but also the plant and animal life in the Bay and ocean. The makeup of the existing sewerage system is shown on Plate I-2.

Of these realities, a complete knowledge of the patterns of rainfall is vital to assigning sizes and, hence costs to each wet weather component. The 62-year record of rainfall measured unfortunately only at one point in San Francisco has been statistically analyzed as one of the realities on which this plan is based. The limited data for other locations in San Francisco was similarly analyzed.

To determine required volumes of storage facilities, and to quantify the overflowed amounts, the incidence of rainfall was examined on the basis of "storm events", using 0.02 inches per hour of equivalent rainfall rate as the existing treatment plant capacity. When this rate is deducted from the continuous hourly rainfall record, the remainder is what overflowed into the receiving waters. A 62-year tabulation of these overflows has been analyzed to determine that the average number of overflows is 82 per year, lasting an average of 2½ hours each, and overflowing 13.88 inches of equivalent rainfall per year on the average.

The area tributary to the system component under consideration is the second physical constraint vital to the sizing procedure used for both volume and rate of flow considerations. Plate I-7 shows the tributary area for the City's major wet weather overflows.

Land usage has been the third vital link to our study in terms of the amounts and types of materials to be treated or emitted into the receiving waters during wet weather periods.

Present land use distribution for the City's three major tributary areas and the City as a whole can be summarized as follows:

TYPE OF LAND USE	PERCENT OF GROSS AREA*			
	NP	RS	SE	TOTAL SFO
Industries & Manufacturing	15	2	13	10
Commercial, Transp. and Utilities	16	4	4	8
Residential	39	56	43	46
Public Lands & Others	30	38	40	36
TOTAL	100	100	100	100

*Plate I-7 defines these areas.

The fourth set of physical constraints concerns the topograhic features of the service area and the downstream drainage course. These features, in the case of gravity-induced flow, govern the speed of mass transport, time of concentration of flows, the direction of flow and hence the selection of the Bay or Ocean as the receiving water for discharge. Plate I-8 shows San Francisco relief photograph.

The final and most delicate constraint which must be considered concerns the water based eco-system. Any discharge from the City must ultimately be put into the receiving waters surrounding the City. These waters are subject to the ebb and flood of the tide, the various oceanographic seasons, the volume of Delta outflow and all other waste discharges to the Bay and its tributaries. Delta outflows are subject to control and probable reduction as a result of increased water needs of the State. The volumes discharged to the Bay will increase as populations expand.

All of these occurrences will have an effect upon the biota of the Bay. The determination of the location of disposal and the degree of treatment to be required will depend upon the existing conditions including the receiving water beneficial uses, the indigenous biota that may be affected and the physical changes that may be anticipated.

The above described conditions represent parameters which are considered to be non-controllable. The following comprise the factors which are considered to be subject to modification by means available to man.

There are five basic components which make up San Francisco's combined sewer system, as follows:

(A) Side Sewers. There are 2000 miles of side sewers which service the off-street tributary areas. This area amounts to approximately 65% of San Francisco, but it only sheds approximately 50% or 4.4 billion gallons per year of the incident rainfall as runoff due to ground absorption, ponding and other losses. Side sewers transport the major fraction of sanitary waste constituents to today's system as opposed to **the horse-transportation era when, it is assumed, significant amounts of animal wastes entered the system from the streets during storms.** Even today street washings are a significant factor **in combined overflow.**

(B) Culverts. There are some 200 miles of culverts which service the 25,000 catchbasins which in turn receive gutter flushings from 845 miles of San Francisco streets. This street sidewalk surface amounts to approximately 35% of San Francisco's 25,000 acre service area, but it accounts for 50% of the runoff (4.4 billion gallons per year) as the perviousness of these areas is highly limited.

(C) Main Sewers. There are two categories of main in the San Francisco sewerage system. They are the collecting sewers and the transport sewers. Sewers smaller than 3 feet in diameter, approximately 750 miles, normally function as collecting sewers to conduct flow from individual service areas to the transport sewers network. The transport sewers which are larger than 3 feet in diameter (approximately 150 miles) function to transport flow from the collectors to a point of disposal or discharge.

Approximately 104 miles of transport sewers have been examined regarding hydraulicz adequacy. Of the examined sewers, 46% are inadequate requiring an expenditure of 75 million dollars to make them adequate for a 5-year intensity storm.

(D) Intercepting Sewers. This 35 miles of intercepting system includes diversion structures and lift stations as necessary to delivery the flow from the main sewers to the treatment facilities.

(E) Treatment Plants. There are three treatment plants which have a present hydraulic capacity to pass all flows up to a maximum flow rate of 340 million gallons per day. Physical constraints for each plant are shown in Plates I-9, I-10, and I-11.

With the realities of the existing system described above, the problem situation with regard to hydraulic considerations is as follows:

The average volume of rainfall runoff derived from extrapolation of historical information to cover the sewer service area is estimated at 8.8 billion gallons per YEAR. The rate of runoff produced by the peak 10-minute intensity of 5-year frequency is estimated at a rate of 13 billion gallons per DAY. During peak storms amounts in excess of the present 10 billion gallons per day transport capacity flow down the streets and gutters which become temporary surface water corridors. Any opportunity to delay or stretch out these peak runoffs would reduce the frequency of deep flows in the streets.

The peak rainfall intensities do not persist for more than a few minutes and the limited time available in which to gain control of the high rate situation, is a problem. This storm water runoff, a great deal of which is mixed with sanitary wastes before it enters the sewer mains, is delivered by virtue of the topographics involved, to the receiving waters in 45 minutes or less (Plate I-1^).

During rainstorms the flow for short periods far exceeds the treatment plant hydraulic capacities and this excess flow must be by passed directly to the receiving water until the facilities recommended by this master plan can be constructed. Any means of stretching out the time period during which runoff flow appears at the treatment plants will allow an increased amount of the flow to be treated.

The maximum rainstorm expected with a five-year frequency delivers combined sewage to the treatment plants at a rate of 13 billion gallons per day during its peak. The plants, however, only have a hydraulic capacity of 0.34 billion gallons per day and the difference now overflows directly to the receiving water. During the 46 overflow days in the average year, however, there is a large amount of unused capacity between rainstorms. Expressing this numerically, during these 46 days the total flow to the Bay and ocean from San Francisco is 13.2 billion gallons and the total plant capacity during that period is 15.6 billion gallons.

The intercepting system delivers approximately 30% of the annual average runoff per year to the treatment plants (2.8 billion gallons) in addition to 36 billion gallons of dry weather flow annually. The remaining 6 billion gallons overflows, discharging about 4 billion gallons to the Bay and about 2 billion gallons to the Ocean.

The problem of pollutants is less evident than the hydraulic considerations previously discussed. Pollution is defined as an impairment to one or more of the beneficial uses of the receiving waters to which discharge is occurring and a pollution source must be evaluated as to the quality of the discharge relative to the natural quality of the receiving waters.

The natural quality of receiving waters fluctuates as a result of many upstream conditions, for example, the changes in quality in response to major fresh water outflow variations which may occur naturally as a result of extended dry periods or as a result of major diversions such as contemplated by the California Water Plan. It is beyond the scope of this report to evaluate the quantitative effects of such planned diversions other than to note that any changes attributable to these diversions will be detrimental to San Francisco Bay's receiving capacity.

Beneficial uses currently being impaired as a result of wet weather overflows include the bacteriological quality of the waters for water contact sports, the aesthetic conditions due to turbidity and discoloration and floating particulate matter and probable localized ecological stress.

While extensive consultant work has been completed to characterize and quantify the pollutant masses contained in combined sewage flows, there are still facets on which technology has not been able to provide the answers. The effect of the quality factors upon the biota has been characterized to a much greater degree. General characterization in terms of total masses of pollutants emitted per storm for a limited number of constituents is available.

Plate I-13 summarizes the available data on an annual basis. However, the available information includes little or no quantity documentation of heavy metals, pesticides, toxicity, turbidity or discoloration.

Acknowledging the limitations of existing quantification and using the available data, it can be seen from Plate I-13 that the annual quantity of gross pollutants attributable to combined sewer overflows is small as compared to the dry weather waste loadings. This degree of difference does not exist during rainstorms and in this regard the waste loading problems are similar to the hydraulic loading problems discussed above. However, it must be remembered that the long-term

contribution of untreated combined sewer overflows is equivalent to the discharge of about 10% of the dry weather mass loadings. It is also significant that combined sewer overflows from the City's system contribute fewer pollutants (for the parameters listed) than that the equivalent separate storm water runoff system would contribute. This is a result of the design of the existing system which captures for treatment about 30% of the total runoff.

This system affords the opportunity to capture fractions of such constituents as precipitated air pollutants, street washings and other discharges presently uncontrolled by separate systems.

To assess the water quality changes attributable to population growth, the pollutants discharged must be quantified and brought into perspective relative to all other sources of water quality change.

Ecological criteria for the disposal of waste effluents have been developed based upon dry weather chlorinated primary effluent.

These are:

1. Dilutions in shallow shoreline water are to be not

less than 100:1 for over 24 hours at a time.

2. The benthos in areas of gravid crab habitation shall not receive sustained exposure to dilutions less than 500:1.

3. Plankton and fish are not to be exposed to dilutions less than 100:1 for a 24-hour period or 200:1 for a longer period.

With the characterization of the discharges developed as outlined above and delineation of the physical constraints of the receiving waters, levels of treatment necessary can be developed for each discharge.

Continuing Projects and Status

There has been a series of continuing projects, the development of which has contributed heavily to this report. Many of these projects are not completed in all phases at this time but sufficient information has been available in either preliminary form or in the format of progress reports to give direction to this report and to delineate the areas of major endeavor that will be required to develop detailed designs for the selected scheme of control. Each of the following projects will be continued to completion.

Hydraulic-Hydrological Data Acquisition System

This project was initiated in 1970 with the retention of consultants, under the direction of the Bureau of Engineering, to design a system for the automated monitoring of rainfall and sewer flow levels.

Bids were advertised in August 1970 for the installation of a central data processor and recorder, 120 remote stage monitoring stations, and 30 remote raingaging sites. Installation and testing is in progress at this time.

Basic equipment consists of a small 16,000-word capacity computer with peripheral hardware and software to serve as the data acquisition central station, tipping bucket raingages at selected locations and differential pressure monitors located at selected critical points within the sewerage system. Data is telemetered from these remote sites to the central computer for initial processing and recording for further analysis on the City's large computer system.

Installation has progressed to where some data from the 1970-71 winter period was available for analysis prior to the publication of this report. Preliminary findings are discussed in Chapter V. This system coupled with the other on-going projects will provide the basis-in-fact for the detailed engineering design of the wet weather control system and could also serve as the nucleus of a final automated remote sensing and control system. At least 5 years of additional data collection and analysis must be continued to establish a basis for systematization and interconnection of isolated components into an integrated system which would allow automated transport and treatment thereby greatly increasing the system's efficiency.

Mathematical Model Development

In conjunction with the above system a mathematical model of the City's sewerage system is being developed. Additionally, the model output will be compatible with the model of

San Francisco Bay developed in 1968 under the Bay-Delta Program such that the effects of various control systems upon the Bay receiving waters can be evaluated. The completed model will include an evaluation of the quantity of various pollutants being transported through the system.

Presently, the model is in a preliminary stage of development without field verification. It is planned to gather field data from various locations to "tune"and verify the complete model. Data collected from the Hydraulic-Hydrologic Acquisition System will provide a wide base of real information for use in verifying the model.

Baker Street Dissolved Air Flotation Facility

The first phase of this project which began in 1968 has been completed. Characterization studies, pilot plant evaluation, prototype construction, and preliminary prototype evaluation have all been completed. The results of these efforts are contained in the appendices to this report. Continuing work will be in the area of further evaluation of the facility and process under actual operating conditions as a potential wet weather treatment process.

Bay and Ocean Effluent Disposal Studies

Oceanographic and ecological studies of the Bay and Ocean are now drawing to a close. These investigations included oceanographic monitoring and ecological studies over three seasons in the Gulf of the Farallones and in San Francisco Bay. Physical measurements included currents, mass water movements, water quality and surface drift studies.

Tidal exchange ratios were also determined for the mouth of the Bay in cooperation with the SWRCB, DWR, Dept. of Fish & Game, USACE, USGS and the RWQCB. Ecological investigations included plankton studies, benthic studies, diving investigations, intertidal studies, on-site cage experiments and extensive laboratory bioassay investigations of a wide range of organisms in both long term and short term experiments. Experiments included static bioassays with adult fish and invertebrates; environmental chamber studies of invertebrate larvae; continuous flow bioassays of Dungeness crab larvae; microcosm studies with Dungeness crab; blood studies of stickleback and biostimulation investigations.

As a result of these investigations design criteria have been developed for outfalls in the Gulf of the Farallones and in the Central Bay. These criteria fit into three classifications: oceanographic currents, water density and factors which affect outfall performance; ecological criteria which define the conditions for discharge such that there are no harmful effects upon the marine environment; and physical criteria such as design flows and discharge head required for outfall performance. The detailed results of these studies are contained in the report attached as an appendix to this report and are discussed in Chapter V of this report. Monitoring of Bay and Ocean physical and ecological parameters will be continued hopefully in conjunction with similar monitoring by adjacent counties.

Pilot Plant Operations

In conjunction with both the dry weather and wet weather programs various advanced waste treatment processes must be evaluated. Preliminary work has been completed for the treatment of dry weather flows to attain various levels of goals stipulated by the RWQCB. Details of this effort are contained in the appendices to this report. Further study and evaluation of various residue handling and disposal processes will be required upon adoption of specific limits by the RWQCB. Wet weather treatment processes will also be given further study.

Data Analysis and Evaluation

In conjunction with all of the above work a computerized program of data analysis and evaluation is continuing. Specific programs for rainfall and runoff analyses are being developed to provide the embryonic basis for future control programs.

CHAPTER II

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based upon the studies leading up to the development of this Master Plan Report, the following conclusions can be noted:

1. The combined sewer system when adequately controlled by sufficient storage and treatment capacity provides for a higher level of water quality protection through reduced pollutant emissions to the receiving water as compared to the separate storm and sanitary sewer system. Significant pollutant loadings are contributed from the urban runoff.[1] Separation would result in little or no benefit at great cost. Thus, the City should retain its combined system and develop the requisite control system for pollution abatement.

2. Because of the system of tidal flushing, advective flow passing by San Francisco's shoreline and San Francisco's location at the entrance to the Bay, the goal of total elimination of the degradation of the waters contiguous to the City and attainment of minimal debris and litter deposition upon

[1] ASCE Combined Sewer Separation Project Progress - 10/10/67, p. 6.

the beaches is dependent upon the control of wet weather and dry weather discharges tributary to the Bay and Ocean system derived from 40,000 square miles. Of particular relevance to San Francisco is the effect of the discharges east of Pier 45 upon the beaches west of Pier 45. Studies to date of the water quality at the Marina beach have concluded, "It does not appear that the receiving water quality in the area west of Pier 45 can be improved by managing combined sewage overflows westerly of this point prior to and independent of effective control of overflows and treated effluent discharges easterly thereof."[2] Thus, any complete system for the control of pollution west of Pier 45 must entail control facilities east of Pier 45. Further, wet weather discharges from other agencies must be controlled to a similar degree.

3. Control of 90% of the volumes now overflowing will reduce the present 82 overflows per year to 8 per year and by providing subsequent chemical treatment prior to discharge will provide for a net reduction of total discharges from the City to the Bay and Ocean of the following estimated percentages:

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[2] Reference No. 11, p. II-26.

Constituent	% Reduction from Existing WW Fraction	% Reduction from Total DW & WW Assuming Chemical DW Treatment
COD	52	8
TSS	82	52
Floatables	66	31
HEM	65	15
N	4	0.5
P	9	0.5

The number of days of overflow will be reduced from 46 days per year to 8 days per year and the days of receiving water violations of bacteriological standards reduced from 170 days per year to between 40 and 62 days per year.

4. Persistent toxicants such as pesticides and heavy metals are best controlled at their source. Chemical treatment in some cases can also be used as a method of reduction.

5. Cost-control ratios indicate that the upper limit of control, based upon volume of combined sewage stored and subsequently treated, is in the 99%+ overflow control range which corresponds to a frequency of one overflow per five years.

6. The environmental impact of one overflow per year (a capture of from 95% to 98% of the present overflow volume) should be no greater than that of any lower frequency of overflow (i.e., 98%+ capture average but 2% to 5% loss in the year of occurrence.)

7. A continuing program for monitoring rainfall, combined sewer flows, ocean and bay conditions and electronic data management modeling is required to provide the data base for the

detailed design and operation of overflow control facilities at minimum costs.

8. Wet weather overflow should be discharged away from the shoreline with minimal initial dilution because a surface field has the least chance of affecting gravid crabs. Subsequent dilution should occur as a function of the net seaward advection of the surface waters. This pattern of discharge will protect the benthos and shoreline from adverse ecological effects. Although these surface fields will best protect the crab life cycle, they will not meet the RWQCB criteria for turbidity and color. To meet both the ecological and the aesthetic criteria of the RWQCB will require a significantly higher level of treatment for wet weather discharges than would be required only for protection of the benthos.

9. Discharges south of the Bay Bridge are undesirable and it is unlikely that the ecological criteria can be met by prolonged high rates of wet weather discharge even with the recommended treatment levels.

10. Treatment of wet weather discharges must provide substantially complete removal of gross settleable and floatable material plus sufficient removals to meet the RWQCB requirements for turbidity, color, grease and toxicants. A 99.9% reduction of coliform organisms will be required within

the Bay and a 99.0% reduction for discharges to the Gulf of the Farallones under certain oceanographic conditions in the early fall and late spring.

Recommendations

The following recommendations are based upon the need to adequately protect the beneficial uses of the receiving waters contiguous to the City from impairment, due in part to combined sewer overflows from the City and County of San Francisco, and based upon the foregoing conclusions of all studies to date regarding the City's wet weather problem:

1. The concept of constructing combined sewers within the City and County of San Francisco should be retained and continued in all future sewerage facilities construction in the City. The combined system with the following controls upon overflows, represents the most secure system of water pollution control that can be reasonably built.

2. Control facilities should be constructed to provide sufficient storage and treatment capacity so that no more than 8 overflows will occur in each year. This design point represents the control of up to 90% of annual combined sewer overflow discharges and if data acquisition confirms possible reduced size requirements from present estimates, additional funds may not be required to provide any further environmental protection

as equivalent protection can be obtained by more efficiently managing the basic units of this system.

3. All discharges of combined flow should be given a level of treatment sufficient to protect the most stringent beneficial uses now recognized. In particular, persistent toxicants and floatable materials must be eliminated and pathogenic organisms must be reduced to levels to provide for the maintenance of water contact sports standards on the beaches and in the Bay at all times. Solids discharges should be minimized if not eliminated. All toxicants must be reduced to levels attainable by source control.

4. The recommended plan for implementation is as shown in Chapter VI of this report and has been prepared so as to best attain the following criteria:

(a) That the treated waste be discharged to the Bay or Ocean through properly designed outfalls so as to have no adverse effect on marine life, the water, or beaches.

(b) That treatment rate can be varied to meet special flow or available dilution changes.

(c) That there be flexibility to meet changing water quality requirements and needs for reclaimed waste water and a "building-block" concept is included to minimize premature abandonments due to changing plans.

(d) That direction of the City Planning Commission, the Bay Conservation and Development Commission and other agencies be reflected to avoid adverse effects on the future development of San Francisco, particularly waterfront or water areas and that use of valuable property for treatment facilities be avoided.

(e) That valuable land such as Golden Gate Park and the north waterfront area be released from sewage treatment use as replacement facilities with multi-use potential are constructed in more appropriate locations.

(f) That financing of the plan implementation be feasible and recognize increasing maintenance and operation costs and the time span relating to San Francisco financing alone or being expedited by Federal and State funding.

(g) That a cost-benefit relationship be included so that policy on the degree of wet weather treatment can be established.

(h) That immediate upgrading of the effluents from the treatment plants can be undertaken.

(i) That substantial reduction in flooding of City streets can be obtained.

(j) That the degradation of receiving waters by combined overflow be substantially reduced.

(k) That a viable industrial waste program be provided to control toxic discharges at the source with supplemental treatment as necessary and technically feasible.

(l) That there be long-range capability for the consolidation of the three treatment plants into one plant.

(m) That an undue investment in facilities need not be prematurely abandoned if it proves necessary in the next century to prohibit all discharges into the Bay.

(n) That there be capability to effectuate an agreement for San Francisco to accept effluent from agencies in northern San Mateo County to facilitate a regional consolidation plan.

(o) That there be compatability with the anticipated Bay Area regional sewerage plan.

(p) That there be capability of conversion to rail transport of solids (dried sludge) in the event a local or regional rail haul plan for solid waste is implemented.

(q) That advantage be taken of the City's hilly topography for underground storm storage.

(r) That there be direction toward a central control system so that dry weather flow, wet weather flow and street drainage can be managed with high speed decisions on assignments of flow increments to varying transport and treatment facilities to make the maximum use of available capacity with changing storm patterns.

5. The City should provide capacity to collect the runoff from all areas within its bounds with appropriate reimbursement to the City for the costs incurred in the collection of runoff from Federal and State lands. Agencies outside of the City's bounds should also be allowed to purchase capacity within the City's system.

6. A program for the collection of all minor runoff outlets on the City's periphery[3] should be initiated.

7. The wet weather treatment capacity deficit now existent should be alleviated through the construction of a major facility at a site in the vicinity of Lake Merced. This plant would provide for a minimum of chemical treatment for dry weather flows and provide split flow options for wet weather flows consistent with required effluent quality. This site is optimum for ocean disposal, waste water reclamation via irrigation, groundwater recharge or discharge to Lake Merced for recycling as well as for sub-regional consolidation with smaller coastal facilities in San Mateo County if cost studies warrant.

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[3] Shoreline Outlet Survey, 1965.

CHAPTER III

REGULATORY AND ADVISORY AGENCIES

The list of agencies now involved in waste water regulation has grown in the last year to include the State Water Resources Control Board acting through the Regional Water Quality Control Boards, State agencies such as the Department of Fish and Game acting both independent of and through the Regional Water Quality Control Boards, the State Department of Public Health, the Bay Conservation and Development Commission, the Environmental Protection Agency, the Bureau of Sport Fisheries, and the U.S. Army Corps of Engineers. The Regional Water Quality Control Board adopts requirements for waste discharge based upon the recommendations of the other agencies.

First concerns in the past were with the dry weather discharges of sanitary and industrial wastes and were directed toward the basic parameters such as maintenance of dissolved oxygen levels and the prevention of gross pollution due to solids discharge and massive discharge of toxicants. Discharge requirements today not only concern the aspects of gross pollution, but also biostimulants, chronic toxicants, and aesthetic degradation.

Existing Discharge Requirements

The San Francisco Bay Regional Water Quality Control Board has established requirements for each of the City's three water pollution control plants. These requirements apply to all discharges from the plants during both wet and dry weather, and are based upon the necessary effluent quality to protect the beneficial uses of the receiving waters that might be affected by discharge from the plants.

These requirements are set forth in Appendix III as they are presently effective. It is anticipated that the Regional Board will establish revised requirements in connection with the North Point Plant during the fall of 1971 and the Board may change its requirements on the other San Francisco outfalls from time to time in the future.

Other Requirements and Conditions, North Point and Southeast Plants

The Board has required the City to submit a preliminary engineering report and cost estimates for facilities needed to comply with the following numerical ranges:

Reduction in receiving water turbidity	5 to 30% in 90% of the determinations made on any day in the area of greatest turbidity.
Floatables in the receiving water at any place	10 to 50 mg/sq meter.
Grease in the effluent	5 to 30 mg/l.
Settleable Matter	Those objectives listed above.

A firm and detailed time schedule for all investigations necessary to implement a program to minimize all discharges of waste which would not comply with requirements prescribed which would result from equipment or power failure.

The Board has also established requirements for the wet weather overflows from the North Point and Southeast sewerage zones and for the effluent from the Baker Street Dissolved Air Flotation Facility.

The requirements are basically the same as those for dry weather discharges and are set forth in detail in Appendix III.

Policy for future waste discharge requirements will be guided by the objectives contained in the Interim Water Quality Control Plan for the San Francisco Bay Basin. Portions of that Plan are as follows:

Water Quality Objectives and Waste Discharge Prohibitions

"It is the intention of this Regional Board to regulate all controllable factors so as to protect the quality of Basin waters from deterioration and to ultimately enhance the quality of all waters. The ultimate protection from the effects of Wastewater will be best afforded by source control of non-degradable deleterious materials, reclamation of all reclaimed portions and relocation of non-reclaimable portions to areas where the environmental impact would be negligible."

"Within the context of the need to implement waste treatment and disposal programs at early dates and the current pertinent studies it is this Regional Board's intention to implement the following water quality objectives and prohibitions. These objectives and prohibitions are designed to maintain or enhance water quality."

"WATER QUALITY OBJECTIVES

No controllable water quality[a] factor shall cause any of the following water quality objectives to be exceeded.

A. TIDAL AND NON-TIDAL SURFACE WATERS

Apparent Color

No significant variation beyond present natural background levels.

Turbidity

No significant variation beyond present natural background levels.

Bottom Deposits

None in measurable concentrations above natural background levels.

[a] Controllable water quality factor means any human activity or natural occurrence which directly or indirectly affects water quality and can be regulated.

Floating Material

None other than of natural causes.

Oil or Other Materials of Petroleum Origin

None floating in quantities sufficient to be visible and none suspended or deposited at any place.

Odor

None other than of natural causes.

Pesticides

No individual pesticide or combination of pesticides shall reach concentrations found to be deleterious to aquatic biota or wildlife or reach objectionable levels in fish or shellfish used for human consumption.

Hydrogen Ion Concentration - pH

There shall be no change in the natural ambient pH value at any place in the main body of the receiving water by more than 0.1 pH unit, nor shall the pH of the waste itself exceed the range 7.0 to 8.5; or 6.5 to 8.5 when the natural ambient value is as low as 6.5.

Biostimulants

None in concentrations sufficient to cause deleterious biotic growths. Whenever natural factors cause such concentrations then controllable factors shall not cause further increase.

Toxic or Other Deleterious Substances[a]

No toxic or other deleterious substances shall be present

[a] Including but not limited to pesticides, heavy metals, materials such as polybichlorinated biphenols and all materials which impart a taste or odor to fish, wildlife or waterfowl flesh.

in the receiving waters in concentrations or quantities which will cause deleterious effects on aquatic biota, wildlife or waterfowl or which render any of these unfit for human consumption either at levels created in the receiving waters or as a result of biological concentration.

Radioactivity

None present in concentrations exceeding levels set forth in California Radiation Control Regulations, Subchapter 4, Chapter 5, Title 17, California Administrative Code.

Temperature

Those objectives prescribed by the State Water Resources Control Board in its "Policy Regarding the Control of Temperature in Coastal and Interstate Waters and Enclosed Bays and Estuaries of California.

B. TIDAL WATERS

Coliform Organisms

Sewage bearing wastes shall be treated to the following levels of quality at all times:

Discharges to any embayment, slough, creek or other confined or shallow waters:

Volumetric Dilution	Quality
Tidal water/waste, at point of access	
Equal to or greater than 100:1	The waste shall not cause the receiving water surface to exceed that bacterial quality

	prescribed in Section 7958, Title 17, California Administrative Code.
Less than 100:1 but greater than 10:1	The waste shall not cause the receiving water surface to exceed a median MPN of coliform organisms not in excess of 23/100 ml as determined from the results of the previous consecutive 7 days for which analyses have been completed.
Equal to or less than 10:1	At some point in the treatment process the waste shall not exceed a median MPN of coliform organisms of 2.2/100 ml as determined from the results of the previous consecutive 7 days for which analyses have been completed, and the waste as discharged shall not exceed the following limits of quality: 5 day 20°C BOD — 5.0 mg/l median; 10.0 mg/l maximum Turbidity — 10 Turbidity units maximum

The Regional Board will consider exceptions to the above coliform objectives for dilutions of less than 100:1 for certain wet weather discharges when it deems that an inordinate financial burden would be placed on the discharger and when it finds that an equivalent level of environmental protection can be achieved by alternate means.

<u>Submerged deepwater discharges in the Ocean:</u>

The waste shall not cause the receiving water at any place being protected for water contact recreation or within 1000 feet offshore from extreme low water[a] to exceed that bacterial quality prescribed in Section 7958, Title 17, California Administrative Code.

The criteria prescribed in the 'National Shellfish Sanitation Program Manual of Operations, Part 1, U.S. Department of Health, Education and Welfare" are the objectives for any area being protected for the taking of shellfish for human consumption.

Dissolved Oxygen

Present levels of dissolved oxygen will be preserved but in areas where oxygen levels are less than the following, the following objectives shall apply to the main body of the tidal waters:

Annual median	80 percent of saturation
Minimum	5.0 mg/l

When natural factors cause lesser concentrations, then controllable water quality factors shall not cause further reduction.

Salinity

Ocean Waters

No significant variation beyond natural background level."

[a] "Extreme low water" means that low tide level which occurs during annual spring tides.

"D. GROUNDWATER

No controllable water quality factor shall degrade the quality of any groundwater. This Regional Board will consider exceptions where the controllable factor is reclaimed wastewater and where existing and potential beneficial uses will be protected.

E. RECLAIMED WASTEWATER

Those quality limits prescribed in Title 17, Section 8025 through 8050, California Administrative Code.[a]

[a] This Board will consider incorporating in this Plan certain reliability criteria which we understand are now being developed by the State Department of Public Health.

"WASTE DISCHARGE PROHIBITIONS

The following waste discharges are hereby prohibited:

A. DISCHARGES TO TIDAL WATERS

1. Any sewage bearing wastewater, regardless of the at any place:

a. Inland from the Golden Gate; within 200 feet offshore from the extreme low water line.

b. In the Ocean; where they will adversely affect waters over rocky substrates or within 100 feet offshore from the extreme low water line and where the waste will not receive a minimum dilution ratio of 100:1 as it reaches the surface.

The Regional Board will consider exceptions from the above prohibitions for certain wet weather discharges and other discharges having high initial dilution when it deems that an inordinate financial burden would be placed on the discharger and when it finds that an equivalent level of environmental protection can be achieved by alternate means.

c. To Tomales Bay, Bolinas Lagoon and Drakes and Limantour Esteros.

2. Any discharge which does not comply with the water quality objectives for tidal waters contained in this plan.

3. All sewage from vessels to Tomales Bay, Bolinas Lagoon, Drakes and Limantour Esteros, Princeton Harbor and within 200 feet offshore from any shoreline or shoreline structure in the San Francisco Bay system.

B. OTHER DISCHARGES

1. Floatable rubbish or refuse into surface waters or at any place where it may contact surface waters.

2. Silt, sand, soil, clay or other earthen materials from mining, construction, agricultural, lumbering or other operations in quantities sufficient to cause deleterious bottom deposits or turbidity or discoloration in excess of natural background levels in surface waters.
3. Oil or materials of petroleum origin in quantities sufficient to be visible.
4. All sewage bearing wastes to non-tidal waters. This Board will consider exceptions where a discharge is approved as part of a reclamation project or where an alternate discharge location is not possible.
5. All conservative toxic and deleterious substances, above those levels which can be achieved by source control, to waters in the Basin.
6. All discharges of sewage sludge and industrial sludge to waters in the Basin.

BOARD INTENTION TO ADOPT PROHIBITIONS

It is the intention of this Regional Board to adopt prohibitions no later than July 1, 1973 for all waste discharges which have not had substantially all toxicants and biostimulants removed to the following areas of limited tidal interchange:

a. South San Francisco Bay and the Northern and Eastern end of the Bay system.

b. Any embayment, slough, creek or other confined or shallow water area.

The details of the specific areas from which such wastes are to be excluded and the scheduling for removal of existing discharges into those areas will be specified in the prohibitions."

These objectives together with the existing requirements form the basis for evaluating the necessary treatment facilities for wet weather discharges.

Federal agency discharge requirements are at this time unknown. It is anticipated that various guidelines for mass emissions of specific constituents will be forthcoming.

Other Federal guidelines that must be considered at this time are those for the Design, Operation and Maintenance of Treatment Facilities published in September 1970.

Section 601.35 of Title 18 of the Code of Federal Regulations concerns the area of operation and maintenance of facilities; Section 601.36 concerns the design of facilities.

Section 601.36 states that "no grant shall be made for any project unless the Commissioner determines that the proposed treatment works are designed so as to achieve economy, efficiency, and effectiveness in the prevention or abatement of pollution or enhancement of the quality of the water into which such treatment works will discharge and meet such requirements as the Commissioner may publish from time to time concerning treatment works design so as to achieve efficiency, economy and effectiveness in waste treatment."

The EPA will use the guidelines to evaluate the compliance of any proposed project with the above noted regulation. Following are significant quotes from those guidelines.

"Planning for the proposed project must take into account all aspects of environmental quality protection."

"Due consideration must be given to the advantages of regional and basin sewerage facility planning. Whenever feasible, municipalities should join together in cooperative regional treatment systems, composed of one or more treatment plants depending on water quality requirements and economic, operational, and other appropriate considerations."

"Where regional waste water management plans have been developed and approved by an appropriate agency, the project should conform to such plans."

"Any proposed project must be designed and reviewed in light of the entire waste treatment system"

"The engineering report shall specifically indicate the anticipated removal efficiency of BOD, suspended solids, and other appropriate parameters, and the total pounds of BOD, suspended solids, and other significant constituents to be discharged per day."

"Provision for ultimate disposal of sludge must be clearly indicated and must be in accordance with interstate, State, and FWQA requirements. It is not sufficient merely to indicate such processes as drying beds, vacuum filters, or incinerators, without also describing the method to be used for final disposal of the sludge cake or sludge residues."

"No sludge residues, grit, ash, or other solids may be discharged into the receiving waters or plant effluent."

"Sludge elutriation is not considered desirable and will not be approved without adequate safeguards."

"The facility should be capable of operating satisfactorily during power failures, flooding, peak loads, equipment failure, and maintenance shutdowns. A minimum of primary treatment should be provided at all times. Disinfection and higher degrees of treatment may be required where necessitated by the uses of the receiving waters."

"In systems handling only dry weather flows, the incorporation in the design of mechanisms for bypassing treatment plants or pumping stations must be avoided if at all possible. Any exceptions must have prior approval of the State and FWQA."

"Where incorporation of bypassing facilities is necessary, considerations must be given to separation of combined systems, detention facilities, or other alternative means of control or treatment, and disinfection of overflows."

"Where necessary, pilot plant studies should be made to determine the final design criteria for the treatment facility."

"The plant site must be sufficiently large to permit expansion of the facility to provide for foreseeable future needs, such as increased capacity and higher degrees of treatment."

"<u>Combined Sewerage Systems</u>

The problem of pollution from combined systems shall be considered in early project planning. Possible solutions, both short and long term, shall be outlined in the engineering report. Consideration shall be given to detention facilities and disinfection, separation of combined systems, treatment or control of overflows or other solutions.

"Discharges in close proximity to shellfishing beds, public water supply intakes, or contact recreation areas should be avoided. Where such discharges are unavoidable, special precautions must be taken. In addition to the items listed above, the following are recommended and may be required:

(a) Dual chlorination units.

(b) Automatic facilities to regulate and record chlorine residuals.

(c) Automatic alarm systems to give warning of high water, power failure, or equipment malfunction.

(d) Sand filters or polishing ponds following secondary treatment."

"Plant and upstream bypasses should not be permitted."

"Exceptions, even for combined systems, shall not be considered until every effort has been made to minimize the discharge of untreated wastewater to waters by utilizing detention facilities or other alternative means of control or treatment, disinfection of overflows, separation of combined systems, and correction of excessive infiltration."

"The use of equalization tanks to decrease the impact of peak loads is recommended."

"Baffles or other means must be provided across the surface of primary tanks, secondary tanks, and chlorine contact tanks to prevent the discharge of floating materials."

"All final settling tanks must be provided with skimming devices to collect and remove floating solids."

With regard to the effects of Regional Plans upon the City's wet weather control plans, the following two statements serve to illustrate the City's position.

First is a quote from the Bay-Delta Program Final Report of 1969:

"The cities of San Francisco and Sacramento have combined storm and sanitary sewer systems. During storm periods these systems do contribute significant pollutional loads to the receiving waters. The solution to this problem lies in either separating the storm and sanitary sewers or in storage and/or treatment of of the combined overflow during storms. Both cities are currently engaged in making studies to determine the best course of action. Whatever the solution, it will be achieved on a local basis. It must be emphasized, however, that control of pollution from combined sewers is necessary to maintain water quality objectives."

To date there has been no indication of any change in the State's position regarding wet weather control. The City however, will not neglect the regional aspect of the problem. Our concern in this regard is contained in Resolution 558-69 of the Board of Supervisors which states in part:

> "RESOLVED, That the Board of Supervisors expresses its willingness to explore and pursue, independently and in cooperation with appropriate regional agencies, all economically feasible methods of meeting applicable Federal and State water quality requirements and standards, including the regional concept expressed in the Bay-Delta Report and local treatment and disposal alternatives, in order to provide for water quality meeting all applicable standards at the lowest cost to the residents of San Francisco."

CHAPTER IV

EXISTING CONDITIONS

Introduction

To define and develop a master plan for the flow from the City which contains variable fractions of materials derived from rainfall, runoff, industrial wastes, sanitary wastes, and the domestic water supply, the relative amounts of each contributary stream had to be quantified.

Ostensibly all of the sanitary and industrial wastes receive treatment in a separate system that has capacity to handle wet weather infiltration while no treatment is afforded the remaining runoff. In a combined system some fraction of each stream receives treatment. In San Francisco the net annual emissions from combined sewer overflow from the existing system is equal to or less than that which would have resulted from the runoff from a separate storm water system. The following discussion of existing conditions will illustrate each component as shown in Plate I-2 and will amplify the source data of Plate I-13.

Existing Sewer System

The existing combined sewerage system must convey, treat and dispose of both dry weather flows and wet weather flows. To function under the extreme ranges encountered under the dual regimen of wet and dry weather conditions, the system has been arranged and sized to accomodate the wet weather flow conditions with little regard to the problems of dry weather flow conveyance.

As previously noted, the present dry weather treatment system does accommodate approximately 30% of the annual storm-water runoff in addition to the dry weather sanitary flow. The worth of this total system in 1971 dollars is estimated to be $1.33 billion.

Service Area

The land mass of San Francisco amounts to 28,000 acres and the public sewer system services an area of 24,000 acres. The remaining areas maintain private sewers, the sanitary flows of which are discharged into the public system.

The Federal Government System includes:

Presidio	1134 acres	
Ft. Mason	63 acres	
Hunters Point	522 acres	
Veterans Admin.	36 acres	
TOTAL		1755 acres

The Park Rec. Dept. System includes:

Golden Gate P.	1,010 acres	
Lincoln Park	155 acres	
Mc Laren Park	318 acres	
Candlestick P.	77 acres	
TOTAL		1560 acres

The Port Commission System includes:

Other than streets	334 acres	
Streets	212 acres	
TOTAL--------------------------		546 acres

Private Corp's includes:

P.G.& E. - Hunters Point	23 acres	
Bethlehem Steel	30 acres	
San Fe RR	60 acres	
TOTAL--------------------------		113 acres

Within private sewer service areas there are many shoreline outlets,[1] most of which are small. Uncollected they represent a potential pollution problem associated with the surface debris and drainage resulting from the activities in the area. This includes such hazards as might result from industrial spills of oil or toxic materials.

The sanitary service area therefore includes the 28,000 acre land mass of San Francisco and an additional 1990 acres in San Mateo County.

Of the total service area within the City there are 13,000 acres at an elevation greater than 50' (City datum). There are also 2300 acres of subsidence area as shown on Plate IV-1.

The City is divided into three sewerage service areas, each of which is presently served by a primary sewage treatment plant.

(1) Shoreline Outlet Survey.

The three plants are the North Point Water Pollution Control Plant, Richmond-Sunset Water Pollution Control Plant, and Southeast Water Pollution Control Plant. The location of each of the plants and its outfall, together with the approximate area it serves, is shown on Plate IV-3. The total service area for each plant also consists of various land uses and population densities which are shown for each major treatment-drainage district and San Francisco as a whole on Plate IV-2.

Existing Combined Sewer System

As was noted earlier, San Francisco's combined system is designed for the conveyance of wet weather runoff flows from the City to the receiving waters with minimal incident flooding and public inconvenience. The system starts with the individual property drains.

Private Property

Inlets in private property consist of yard drains and roof drains to collect rainfall runoff which is discharged jointly into the main street sewer along with the sanitary sewage from the property in a single side sewer. These plumbing requirements are controlled by Section 308 of the Plumbing Code. The interconnection of storm and sanitary flows thus begins in the private system of each property owner in the City.

There is provision for separate drainage facilities in areas where separated storm sewers are existent. Section 815 of the Plumbing Code which requires two side sewers be constructed from each house in separate sanitary storm districts will be superfluous upon completion of the recommended control facilities.

Street Drainage

Rain that falls on paved street areas collects in the gutters which are sloped to drain into either stormwater inlets or catchbasins usually located at each intersection. A change in practice is planned whereby the City Standard Storm Water Inlet will be generally used for new construction instead of the catchbasin.

The sewerage system in San Francisco has approximately 25,000 catchbasins. Unlike most cities of the Bay Area, it has been the practice up to the present time to install catchbasins with traps instead of stormwater inlets without traps to pick up stormflow from the streets. The factors that have been used to justify this practice are:

1. The physical shape of the catchbasin tends to trap sand and debris and large materials and ostensibly prevents them from entering the sewers. This practice may have been justified prior to the introduction of the automobile and the practice of paving streets. Sewer plugging problems today are related to the subsidence areas of the City where low velocities and sewer subsidence have combined to provide traps for solids in the sewer.

2. The use of a trap can provide a water seal to control odor. However, during the dry season the water evaporates and the seal is lost in most cases. Random field checks of 725 catchbasins in 6 districts indicated more than 45% were too full of debris to determine whether a trap existed or not. Of the remaining 468 catchbasins, 30% had no trap. Odors were observed in only 3 catchbasins. In the same area 38% of the catchbasins that had a trap had no seal and 16% of manholes tested had an odor. In the catchbasins that had an odor (1%), the odor does not appear to related to the seal condition.

3. The use of catchbasins with traps was probably once believed to provide for rat or other rodent and vermin control.

While we have no data to either substantiate or contradict this belief, intuitive logic does not support it as rats can burrow and swim. Further, the previously noted lack of any seal on many catchbasins has not resulted in any known rodent problem.

Sewers

The sewers existent excluding side sewers and culverts were inventoried in 1964 as follows:

Type	Diameter	Type of Con- struction	% of Total	Miles Located in Subsidence Areas Not on Piles *	Total Miles in Place
Collecting sewers	8" - 36"	princi- pally VCP	82.5%	34.1 miles	716 miles
Transport sewers	36" - 60"	56% brick, 44% RCP	11.7%	15.96	102
Transport sewers	60" & Larger	13% brick, 87% RCP	5.8%	0.79	52

* Expensive to maintain

Over 80% of these sewers are over 33 years old. The various age groupings of the City's sewers can be described as follows:

Period	Age	Total Miles in Period	% of City Total
Prior to 1892	79 yrs & older	250 miles	29%
1892-1905	66 to 79 yrs.	79 miles	9%
1905-1938	33 to 66 yrs.	374 miles	43%
1938-1964	7 to 33 yrs.	166 miles	19%
		870 miles	100%

For a plane of reference, Plate IV-4 illustrates the inter-relationships of the components of a combined sewer system; Plate I-7 describes the points of discharge, and the tributary sewers mains and the areas serviced; Plate IV-1 describes the existing sewers that are supported on piles and the area within which the system is subject to subsidence.

Design Criteria

San Francisco's sewers were installed under one of the following methods:

(a) Under private contract, approved by the City;

(b) Under Assessment proceedings; or

(c) Under public contract.

Standards of design for sewers installed prior to 1952 were less sophisticated than those in use today. A review of plans and specifications prior to that time indicates that in general sewers were installed 8 to 12 ft. deep and on a straight line grade. There appears to have been little or no consideration for velocity, energy, and momentum of the flow of these sewers. The designs were probably made solely upon the drainage calculation for required size and slope. In 1952, a set of subdivision regulations were published for the information and guidance of all subdividers, engineers, and surveyors with reference to subdivision of land within the City and County of San Francisco.

Beginning in 1956, hydraulics began to play a greater part in the design of the sewer system. However, 850 miles of sewers were already constructed and were based upon the limited design methods. The newer designs were not merely based on a hydraulic gradeline but energy considerations and momentum considerations were analyzed and incorporated into the design. Feedback from field measurements to verify design intent is currently being pursued. When this final verification is completed, further improvement to current design procedures will be initiated.

There are transitory cause and effect relationships in the system functioning unpredicted by the traditional steady state analyses utilized when manual calculation is involved. Sewers whose flows are substantially more or less than predicted by their designers were observed during the 1970-71 winter's field measurements and must be sequentially investigated, quantified and converted into revised design procedures and computer programs.

The design of the sewer system is dependent upon several major factors which are not considered controllable. These factors are; the tributary area, the runoff coefficient of that area, the topography of the land, the slope of the terrain, direction of the slope of the terrain, the amount of rain that falls and the land use in the various districts. The area tributary to each outfall and the quantities of flow derived from those areas relative to the assumed runoff coefficients have also been presented. Examination of the inventory of sewer capacities through the upstream reaches of the system shows zones of the inadequacy of transport capacity. Plate IV-6 indicates the location of the major sewers with inadequate capacity, and Plate IV-7 summarizes the cost of the correction of these deficiencies to attain the City's 5-year frequency design storm capacity.

As can be noted in Plate IV-8 different outlets have different average transport velocities with the average velocity in the Richmond Sunset district substantially higher than the average velocity in the other districts which accounts for the smaller flow/size ratios of the western discharges. The eastern grade is broken at approximately 1/2 the distance to the outlet which requires substantially larger sizes to transport the flows at the lower velocities available to this area. This situation produces ancillary problems which negate the apparent flow capacity benefits of the larger conduits. As the transport velocities decrease, the heavier solids carried by the higher velocities upstream deposit in the sewer. These deposits elevate the hydraulic surfaces and cause flooding well before design capacities are attained under storm conditions. This situation is a critical flood control factor in the three areas denoted in 1869 as swamp areas. Due to the level street surfaces in the lower zone and the practice of matching inverts at the junctions, the interconnected mains are subject to flow reversal as a result of daily variations in flow conditions between the transport and collecting sewers. This phenomena results in septic deposits in head ended or collector sewers located in areas with limited available grade with frequent plugging and the associated odor problems.

Diversion Fittings

There are various types of diversion fittings used on the transport sewer system to regulate flow in one conduit or divert it from one conduit to another.

The overflow (weir) type fitting functions when the water level reaches a predetermined elevation, it overflows the weir into an auxiliary system or another parallel system to be transported to a point of discharge. The weir may be a side weir, and end weir or a leaping weir in the bottom of the structure.

The underflow type of fitting can be constructed with a fixed opening or with a gate to vary the size of the opening to control flow emanating from one system and entering another. This fitting diverts sanitary flow and a portion of the storm flow to the nearest treatment plant via an interceptor sewer. The amount of flow for which diversion fittings have been designed to retain is approximately equal to 3 times the average dry weather flow.

Interceptor Sewers

The interceptor sewers located along the shoreline as shown on Plate IV-9 are designed to transport 100% of the dry weather flow from the diversion fittings beginning in most instances at levels below tidal elevations. As a result, 90% of the intercepted flow is transported through treatment to ultimate discharge via pumping stations. Failure of any of the 17 pumping stations, not including pump stations at treatment plants, results in the overflow of raw sewage. Failure of pumping stations at treatment plants have the same result, with the exception of the booster pump station at SEWPCP which pumps effluent through the outfall under conditions of high flow.

Pumping Stations

Where there is insufficient elevation available for the gravity transport of flow through the interceptor system to the treatment plants, pump stations are utilized to provide the energy to restore the fluid to an elevation where the flow then can proceed by gravity to the plant. There are 22 pumping stations located in the City, the 14 major pump stations are as shown on Plate IV-9. Of the 17 stations, 11 are on the Bay side and 6 are on the ocean side of the City. These stations intercept the flow from the 6,450 acres and have a combined capacity of 76 MGD. A detailed tabulation for each station including the existing deficiencies is shown in Plate IV-10.

Outfalls

The quantity of flow in excess of the amount diverted to treatment will bypass into the outfall system.

There are two types of shoreline outfalls in the San Francisco sewerage system. The first is one that discharges below tide level. The second is one that discharges above tide levels typically over a beach. If the outfall discharges below tide level, there is no need to put a structure at the end to prevent children or stray animals from entering the sewers. The device utilized is a gate or weir to control the entrance of tidewater into the system. Under this condition the gate or weir is normally included in the diversion structure upstream. If the outfall structure is above tide level and discharge is over a beach or at the water's edge, it is normal to put an outfall structure at the end. The outfall structure incorporates such features as a gate or a series of gates to pass the flow and perhaps an emergency overflow weir for greater than design flow intensity. The physical shape of each structure varies depending upon the location and the amount of flow and the depth of the outfall.

In general, the gates are automatic flap type gates either rectangular, square or circular in shape. They are placed in the structure in such a manner that invert of the gate is at the approximate elevation of -7' City Datum. This elevation was chosen in order that the invert of the gate be above mean sea level which is -8.6' relative to City Datum. The reasoning behind this choice was that the gate would be out of the water more than half of the time and repair and maintenance could be made to the structure at such times.

The weirs in a structure are set at varying elevations depending upon the location of the sewer outfall. They vary from approximately -4.2 to -3.5 on the Bayside of the City. There are several structures in the City that rely solely upon weirs to discharge flow into the receiving waters having no need for a gate by which to control the flow.

Treatment Plants

During the average year, the interception system directs 36 billion gallons sanitary waste flow and 2.8 billion gallons of the 8.8 billion gallons of wet weather flow to the City's three sewage treatment plants.

Following is a summary of the existing conditions at each of the City's three treatment plants:

North Point Water Pollution Control Plant

The North Point Water Pollution Control Plant was completed in 1951 at a project cost of $8,500,000 and serves an area of 7,500 acres. The tributary area includes residential, commercial and industrial land use areas. Characteristics of the area served is shown in Plate IV-2.

The plant now provides conventional primary treatment consisting of prechlorination, screening, grit removal, pre-aeration and primary sedimentation with chemical coagulation capabilities using ferric chloride, and postchlorination. A flow diagram of the various treatment units and the functions that they perform is shown on Plate IV-11. Hydraulic profiles showing water surface elevations at present average dry weather flow and estimated maximum hydraulic capacity of the of the North Point plant are presented in Plate IV-12.

Bypassing. Flows exceeding the plant capacity are bypassed from upstream diversion structures directly to San Francisco Bay without any treatment.

Preaeration and Primary Sedimentation. Primary settling takes place in six combination preaeration-sedimentation tanks. Each tank is 297 ft. long, including 74 ft. of preaeration, 38 ft. wide with an average depth of 10.7 ft. at a flow of 65 MGD. Total detention time including preaeration at this flow is 2 hours. Detention time, overflow rate and mean forward velocity plotted against flow for each tank is shown in Plate I-9.

Under normal conditions, all six tanks are in operation. About once a year each tank is taken out of service for maintenance and repair.

Chlorination. Chlorination facilities provide for prechlorination of influent sewage for odor control and hydrogen sulfide suppression and for postchlorination of plant effluent for disinfection.

Prechlorination is applied at dosages of between 70 to 90 lbs. per million gallons. A chlorine residual in the effluent of 2.8 to 3.8 mg/1 is maintained to disinfect the plant effluent to the standards prescribed by the RWQCB.

Postchlorination contact time is provided in a 50-ft. diameter tank sized for a contact period of 5 min. at 65 MGD.

Effluent Disposal. The plant effluent flows into an 8-ft. reinforced concrete outfall sewer which branches into two 6-ft. concrete pipes. Each 6-ft. line in turn branches into two 48-in. cast iron outfalls. These four lines discharge the effluent into San Francisco Bay approximately 10 ft. below mean lower low water about 800 ft. offshore. Two outfalls are suspended under Pier 33 and two under Pier 35.

Solids Treatment. The North Point Plant does not include facilities for the treatment and disposal of any of the solids removed during the sewage treatment process. Sludge and scum removed in the primary sedimentation tanks are pumped six miles through a 10-in. diameter force main to the Southeast plant. At the present time the average flow of sludge pumped from the North Point plant to the Southeast plant is approximately 850,000 gpd at a solids concentration of about one percent.

Richmond-Sunset Water Pollution Control Plant

The Richmond-Sunset Water Pollution Control Plant was completed in 1939 at a project cost of $2,000,000 with a design peak wet weather flow (PWWF) capacity of 45 MGD. It has since been enlarged and modified to its present design PWWF capacity of 70 MGD. The plant is located in the southwest corner of the Golden Gate Park, which provides visible isolation from the nearby residential areas. The treatment plant serves a tributary area of about 10,460 acres, the development of which is almost entirely residential. Plate IV-2 shows the area characteristics in more detail.

About 60% of the total flow to the plant arrives by gravity through two main interceptors. The remainder is pumped from the Mile Rock interceptor sewer by the Sunset pumping station to a receiving structure upstream of the plant overflow weir.

Sewage Treatment. The plant provides conventional primary treatment consisting of screening, grit removal, primary sedimentation and effluent disinfection prior to its discharge to the ocean. Solids separated during settling are subjected to two-stage digestion, sludge conditioning and dewatering before disposal as a soil filler within the park.

A flow diagram of the treatment units and their functions is presented in Plate IV-13 . Plate IV-14 shows hydraulic profiles for the flow conditions of present average dry weather flow and estimated maximum hydraulic capacity.

Bypassing. Bypassing in the Richmond-Sunset Plant takes place at two different locations:

(1) the overflow weir of the Sunset pumping station diversion structure in the Mile Rock sewer (weir crest elevation 0.0 ft. City of San Francisco datum) when the flow exceeds the station capacity or upon power failure; and

(2) the overflow weir in the plant headworks bypass structure (weir crest elevation 21.3 ft.) when the flow exceeds plant capacity.

The Sunset pumping station capacity (33 mgd) acts as a throttle during periods of rainfall and bypasses at the Mile Rock sewer overflow weir.

Bypassing at the headworks overflow weir takes place when the total flow through the plant reaches approximately 70 MGD. At this time raw sewage overflows into a 6.5-ft. wide channel, passes through a 4-ft. throat Parshall Flume and enters a 54-in. diameter bypass line that connects to the Mile Rock outfall.

Provisions will be available to chlorinate the headworks bypassed sewage when the present reconstruction of this area is complete. There are no facilities to chlorinate raw sewage bypassed at the pumping station diversion structure.

<u>Primary Sedimentation.</u> Primary settling takes place in five rectangular tanks housed in the sedimentation building. The first four tanks are identical units and were originally built as combination flocculation-sedimentation basins. A fifth tank was added in 1963 and the other four converted to conventional sedimentation basins. At that time, flocculation facilities were removed from the existing tanks. Each tank is now 135.5 by 33.5 ft. with an average depth of 10 ft. Detention time at ADWF of 19 MGD is 2.1 hours.

Detention time, overflow rate and mean forward velocity plotted against flow for each tank is shown in Plate I-10, previously presented in Chapter I.

Under normal conditions, all five sedimentation tanks are kept in operation. Tanks are usually taken out of service once a year for routine inspection and maintenance.

<u>Chlorination</u>. Chlorination facilities are provided for disinfection of the plant effluent. When present reconstruction is completed, chlorination of raw sewage bypassed over the headworks overflow weir will also be possible.

Effluent Disposal. Plant effluent drops into a junction vault and enters the Mile Rock outfall sewer. The 9 by 11-ft. outfall discharges into the Pacific Ocean at the shoreline near the entrance to San Francisco Bay, approximately 7,000 ft. north of the treatment plant.

Solids Treatment. As indicated previously, the Richmond-Sunset Plant is provided with facilities for the treatment and disposal of all the solids removed during the sewage treatment process. Organic solids are first stabilized in anaerobic digestion tanks, then the digested sludge is conditioned by elutriation and coagulant addition, and finally it is dewatered by vacuum filtration and disposed of as a soil conditioner. At the present time the average raw sludge flow to the digesters is approximately 100,000 gallons per day at a solids concentration of about 2.0 - 2.5 percent.

Solids Digestion. Anaerobic sludge digestion takes place in two digesters with a combined volume of approximately 3,200,000 gallons. One tank is 100 ft. in diameter with a fixed cover. Both digesters are provided with external heat exchangers for sludge heating and with compressed gas diffusers for mixing of their contents.

Digesters are normally operated as two-stage digesters with the larger tank acting as the primary digester and the smaller as the secondary. Raw sludge is pumped intermittently into the primary tank at two points which are alternated daily. Both tanks are maintained full, so when sludge is added there is automatic transfer of primary sludge into the secondary digester and of secondary supernatant into the elutriation tanks. Sludge from the primary digester is continually circulated through the heat exchangers and the temperature maintained at about 95°F.

In the secondary digestion tank, transferred primary sludge is allowed to stratify, and, with the exception of a periodic stir-up of the tank contents, the digester is not mixed. Digested sludge is withdrawn from the bottom of both tanks daily and pumped to the elutriation system.

Gas produced during the digestion process is recovered from both digesters and used for mixing and as fuel for the plant steam boilers. Excess gas is burned in a single waste gas burner. Present gas production is estimated to be over 200,000 cu. ft. per day.

Solids Conditioning and Disposal

The final phase in the solids stabilization process involves the preparation of the digested sludge for its ultimate removal from the treatment plant.

Elutriation. Digested sludge and sludge solids recovered from supernatant are washed prior to vacuum filtration in two elutriation tanks to reduce the alkalinity. Each tank is 50.5 ft long by 14.7 ft wide and operates at an average water depth of 9 ft.

Filtration. Dewatering of the conditioned sludge is accomplished on two rotary drum vacuum filters situated on the first floor of the administration building. The filters are 8 ft in diameter by 8 ft long and are provided with Dacron filter media.

Ferric chloride is added in the sludge flocculator just ahead of the filters. Filter cake is collected on horizontal belt conveyors and dumped on a central sloping conveyor which carries the cake to four bins or, when available, directly to a truck. From the bins, sludge cake is loaded into trucks and hauled away. Cake is used mostly in the Golden Gate Park for filling and as soil stabilizer although some is kept available to the public.

Present cake production is approximately 1,200 tons of dry solids per year at an average solids concentration of approximately 25 percent.

Southeast Water Pollution Control Plant

The Southeast Water Pollution Control Plant, completed in 1951 at a project cost of $7,000,000, serves a dry weather flow area of approximately 10,150 acres in San Francisco and 1990 acres of San Mateo County. The plant serves the heavy industrialized area situated in the southeast corner of the City of San Francisco. Reference No. 28 describes in detail the industrial composition of the service area. The tributary area also includes some residential developments. Plate IV-2 gives a detailed description of the service area.

The plant has undergone major modifications in practically all of its treatment units, the latest of which, involving extensive reconstruction work in one sedimentation building, is proceeding at the present time.

The Southeast plant can be more accurately described as two separate treatment plants at a single site. One is a conventional primary treatment plant serving the southeast tributary area and the other provides solids treatment both to the sludge and scum pumped from the North Point plant and to the raw sludge and scum removed at the Southeast primary plant.

Sewage Treatment

The southeast primary treatment plant consists of prechlorination, screening, influent pumping, grit removal, preaeration and primary sedimentation, post-chlorination and effluent disposal. Provision for chemical coagulant additions are being installed.

A flow diagram of the various treatment units and the functions that they perform is shown on Plate IV-15. Hydraulic profiles showing water surface elevations at present average dry weather flow and estimated maximum hydraulic capacity of the Southeast plant are presented on Plate IV-16.

Storm flows in excess of plant capacity are bypassed directly to San Francisco Bay. Bypassing takes place at upstream diversion points.

Primary Sedimentation. Influent normally flows to four combination preaeration-sedimentation tanks. During the time this report was being written two tanks were being modified although one tank was being kept in operation to assure adequate treatment during the construction period.

Each preaeration-sedimentation tank is 262 ft long, 37 ft wide and has an average depth of 11 ft at present ADWF. When modified, the tanks in building No. 1 will be only 247 ft long, with the last 15 ft. being abandoned. Detention time, overflow rate and mean forward velocity plotted against flow for each new tank is shown in Plate I-11.

Chlorination. Chlorination facilities provide for prechlorination of influent sewage for odor control and hydrogen sulfide suppression and for post-chlorination of plant effluent for disinfection.

Chlorine solution for prechlorination is applied to the raw sewage at a rate of between 250 and 300 lb per million gallons.

Chlorine solution for postchlorination is applied at a dosage of between 250 and 350 lb per million gallons.

Effluent Pumping. After passing through the plant, effluent flows into a 6-ft diameter reinforced concrete sewer. The effluent sewer is approximately 2,900 ft long and terminates in the outfall booster pumping station built in 1968.

When plant effluent can no longer discharge by gravity through the outfalls to San Francisco Bay, the level in the pumping station sumps rises until a preset elevation is reached at which time one pump starts at low speed, the pump's discharge line control valve opens and the 30-in. gravity interconnecting line control valves closes. Pump speed changes with flow variation to maintain a constant sump level. When the level drops below the minimum set elevation, the pump discharge line control valve closes, the pump stops, the 30-in. gravity interconnection line valves open and the plant effluent again flows by gravity through the outfall to the bay.

The effluent pumping station was designed to allow pre-dilution by continuous pumping of a mixture of salt water and plant effluent.

Effluent Disposal. From the outfall booster pumping station, the effluent flows through the Islais Creek inverted siphon and into a terminal manhole where the plant outfall begins. The outfall consists of approximately 4,250 ft of 54-in. diameter pipe, 500 ft of which is laid in the transition and offshore sections, and a 300-ft submarine diffuser section. The diffuser section reduces in size from 54 in. to 16 in. and is provided with 18 T-shaped diffusers, each with two lateral ports. The vertical section of each diffuser is about 8.5 ft long and 10 in. in diameter. The laterals are each 4 ft long and 6 in. diameter.

Under existing conditions (no pre-dilution) the minimum dilution in the receiving water is about 50 : 1.

Solids Treatment

As stated previously, the Southeast plant is provided with facilities to treat not only the sewage solids removed during the primary treatment process at the plant site but also the sludge that originates at the North Point plant. The processes include gravity thickening, sludge digestion, elutriation, digested sludge chemical conditioning and sludge dewatering.

Sludge Thickening. Sludge from the North Point plant is discharged directly to the sludge thickening facilities along with that from the Southeast plant.

Sludge thickening facilities consist of two gravity separation type thickening tanks and a thickened sludge pumping station. Each sludge thickener is 91 ft long by 18 ft wide and has an average water depth of approximately 12 feet.

Total solids concentration of the thickened sludge and scum averages approximately 5.5 percent.

Sludge Digestion. The Southeast treatment plant is provided with ten digesters divided in two groups of five tanks, each arranged around a central control building. Each tank is 100 ft. in diameter with a side water depth below overflow of 28.5 ft and is provided with a floating cover. Digesters were originally designed as standard rate tanks, but three of them, tanks Nos. 7, 8, and 9, have beenconverted to high rate operation by the installation of internal gas mixing systems.

Present operation involves the normal use of three high-rate digesters with two standard-rate digesters planned to be converted to high-rate units. These will be available for stand-by and for processing larger chemical sludge volumes. Sludge can also be fed to the other group of conventional quiescent tanks. At the present time all of these five tanks contain large volumes of inactive sludge. Digested sludge is withdrawn to the elutriation tanks.

Solids Mixing. Mixing of the digester contents is accomplished by injecting compressed sludge gas through diffusers located at the tank bottom. Gas is compressed by six rotary type gas compressors, to 18 psi for injection into the digesters. Digester contents are also mixed by the sludge recirculation pumps operating in conjunction with the heat exchangers and the discharge piping terminating at 20 points radially at the bottom around each high-rate digester.

Gas System. Sludge gas produced in the digestion process is metered and then goes to the gas compressor building where it is compressed to a pressure of 28 ounces per square inch. Gas is used for digester contents mixing and as fuel for two steam boilers. Excess gas is burned in four waste gas burners. A gas holder provided in the original installation when heat drying of the sludge was practiced is no longer used.

Heating System. The temperature of the digestion tanks contents is maintained at approximately 95°F. Digesting sludge is circulated continuously through spiral heat exchangers using vertical centrifugal pumps. Hot water provided by steam-to-water heat exchangers is used to heat the spiral heat exchangers. Heated sludge may be returned to only one point in Digester No. 7 and to any of 20 different points in Digesters No.8 and 9. Return points in Digesters No. 8 and 9 are changed in sequence once every shift.

Solids Condition and Disposal

The final phase in the solids stabilization process involves the preparation of the digested sludge for its ultimate removal from the treatment plant.

Elutriation. Digested sludge is conditioned prior to vacuum filtration in elutriation tanks. The tanks are divided into two batteries of four each and are housed in the filtration building. Each tank is 60 ft. long by 16 ft. wide with an average water depth of approximately 12.5 ft.

Digested sludge passes to the elutriation tanks. Although it is possible to bypass the elutriation tanks and process the sludge directly on the filters, this mode of operation is not practiced at this time.

Filtration. Dewatering of the conditioned sludge is accomplished by four vacuum filters located in a large room adjacent to the elutriation tanks. Two filters are the original 8-ft. diameter by 14 ft. long rotary type units rated at 50 tons of solids per day and require an off-time washing process. The other two are larger and newer filters being 11.5 ft. in diameter and 16 ft. long and are continuous cleaned on-line. The latter are coil-type units capable of dewatering 150 tons of solids per day.

Digested sludge is fed to the filter from the sludge storage tank by diaphragm pumps. Filter cake is carried in belt conveyors and stored in an elevated storage bin from which it is trucked away to land fill. Present cake production is approximately 21,000 tons per year at an average solids concentration of 28%.

Waste Water Quality Data

The main waste parameters available over a long-term period for use in this report to document existing conditions are flow, suspended solids, BOD, and grease. Potential parameters of concern for which requirement can be expected at some future time are depicted on the water quality scale as shown on Plate IV-17 through Plate IV-23.

These four parameters are reported in the Bay-Delta Study in conjunction with Total Nitrogen and Total Phosphorus.

The time period used is from July 1969 to June 1970 which is the last complete rainfall period available at the time of writing of this report.

A check of the data shows that the Southeast plant has the strongest influent of the three plants in terms of concentration. Further, on a mass emission basis, North Point is highest, Southeast next, and Richmond-Sunset lowest. This is to be expected noting the industrial nature of the SEWPCP zone.

The data from July 1969 to December 1970 was split into 12-month periods. The first time period was for the 1970 calendar year, Plate IV-24 and the second was for the 1969-70 rainfall year, Plate IV-25. This was done to see if any significant differences were apparent over the period which might be reflected in this split. None was apparent.

The plant data was divided into two additional categories, wet and dry weather months. The wet and dry weather months were split from the 1970 calendar, Plate IV-26 and the 1969-70 rainfall year, Plate IV-27. Again there are no readily apparent significant differences.

In addition to the foregoing long-term data for the parameters of flow, suspended solids, BOD, and grease, data is available from one week of extensive testing at each plant in 1970. Average values for the additional parameters measured are as shown in Plate IV-28. Toxicity measurements as determined by bioassays are shown in Plate IV-29.

Average removal efficiencies are shown in Plate IV-30 for the 1969-70 period or where special measurements were made.

The efficiencies are typical for primary treatment plants.

Water Quality Scale

The aforenoted constituent quality values are illustrative of the existing influent and effluent conditions at each plant for the parameters listed. However, any system of water quality management cannot be restricted to observations of influent and effluent qualities alone. A comprehensive quality scale of comparison is required to provide a basis for the management of the total resources of the system under consideration. This quality scale must start with the quality of the domestic water supply of the system and follow the quality changes through the control system.

Plates IV-17 through IV-23 are the scales now being used. These scales of wastewater parameters have been divided into seven general sub-areas which are as follows:

1. Bioassays
2. Chlorinated Hydrocarbons
3. Chemical and Biochemical
4. Heavy Metals
5. Nutrients
6. Physical
7. Radioactive Substances

The influent and effluent values assigned to the S.F. Water Pollution Control Plants (WPCP) are flow weighted composite data based upon individual plant values measured during a week of testing at each of the three treatment plants during July and August of 1970. There is one exception, the bioassays were actually conducted using a flow composited mixture of influent and effluent from the three WPCP's.

The values assigned as the average San Francisco drinking water data are an average from four sources: Hetch Hetchy Reservoir, Calaveras Pipeline, Crystal Springs Lines and San Andreas Lines sampled in May of 1970. The four sources were averaged because the amount of water from each source varies with the season and the location within the City where the water is delivered.

The Los Angeles drinking water is that supplied by the Metropolitan Water District and it is believed that this water is derived mainly from the Colorado River. The United States Public Health Service limits reported on the charts are either the maximum limit or where no maximum limit is stated the desired limit.

The Regional Water Quality Control requirements are those imposed on the Richmond-Sunset, North Point and Southeast

WPCP by Resolutions 67-2, 70-17, and 69-44, respectively.

Plate IV-20 may be used to evaluate the need for industrial waste source controls for specific constituents, to compare various treatment processes with regard to the full spectrum of constituents rather than only the traditional performance parameters and to aid in evaluating discharge impact on receiving waters. Where data on receiving water background levels is available, it is shown on the scale to represent the discharge condition.

Combined Sewer Overflows

Prior to preparation of this report a basis for predicting flow quantity and pollutant concentration relationships for combined sewages was developed. Studies were conducted during the period of 1966 to 1970 to quantify the dry and wet weather wastewater emissions from a total of six drainage basins in San Francisco. Five of the six drainage basins (Baker Street, Mariposa Street, Brotherhood Way, Selby Street, and Laguna Street Drainage Basins) have combined sewerage systems in which the wet weather discharges consist of sanitary wastewaters, industrial discharges (Mariposa Street and Selby Street Drainage Basins only), and surface runoff wastewaters. A separate storm sewerage system is contained within the sixth monitored basin (Vicente Street Drainage Basin); emissions from this system occur only during wet weather and are not believed to contain sanitary or industrial sewages. The predominant land uses in the six basins are:

(1) Baker: single high value residential.

(2) Brotherhood, Selby, and Vicente: single medium value residential.

(3) Laguna: multiple residential.

(4) Mariposa: industrial.

A total of 20 storms were monitored in these basins and the information was used to develop a profile of the quantity and quality of combined sewage flows in each basin. From the above effort and the results of prior monitoring of dry weather in each basin, it was possible to develop two types of unit waste emission parameters, namely:

(1) Dry weather flow coefficients, based on the per capita emissions of wastewater (expressed as gallons/capita/day)

and constituents of wastewater (lb each constituent/capita/day).

(2) Storm runoff coefficients, for predicting the contribution of storm runoff and system scour on the basis of pounds of contaminant per unit area of the drainage basin.

Evaluation of the Problem

The above coefficients were used to estimate the dry and wet weather wastewater loads on a per-storm seasonal annual basis to evaluate the magnitudes of wastewater emissions under wet and dry weather conditions and to relate this to management program alternatives. Due to the seasonable nature of wet weather emissions as compared to dry weather emissions, two time frames can be selected to examine the impact of combined flows relative to total waste emissions. The first approach is to develop a comparison on an annual basis of the relative magnitude of pollutant emissions from both dry weather and combined sewage flows assuming various levels of treatment for each stream. This approach can be used to provide an overall basis for evaluating the mass of pollutants removed as a function of where and how treatment of each stream is provided. The second approach involves the development of an equivalent day comparison between the estimated dry weather and combined sewage pollutant emissions as a basis for evaluating the immediate impact of combined sewage emissions on receiving water quality.

Annual Basis

The comparison of the relative magnitude of pollutant emissions on an annual basis from the City was developed using dry weather and combined sewage mass emission coefficients for the rainfall pattern of the 1969-70 rainfall year. Assuming that combined sewage overflows occur only at or above rainfall intensities of 0.02 inches per hour as per the system design criteria, then overflows occurred on an average of 2.5 percent of the time on an annual basis whereas the total rainfall time is about 5% of the total year. In the examples developed combined sewage flows generated by rainfall at intensities greater than 0.02 inches per hour were bypassed as combined sewage overflows directly to receiving waters and it was assumed that approximately 65% of the rainfall falling on the City was recovered as storm runoff which entered the sewer system and became combined sewage flow.

The time period investigated for this report was July 1969 to June 1970. Rainfall records from the Federal Office Building were utilized to obtain the pounds of mass emissions due to storms.

The total volume of rainfall reported for the year is very close to the 30-year annual average of 20.78 inches. There was an unusual rainfall occurring in June but in terms of volume and time distribution it appears that 1969-1970 rainfall year was an "average" year. An analysis of the rainfall intensity and duration for the 1969-1970 rainfall year versus the years of rainfall records was not attempted at this time. An assumption was made that the rainfall intensity and duration for the 1969-1970 year would be close to "normality".

The mass emission equations are applied to the volume of rainfall that occurs during any one day. Realizing that the definition of the start and end point of a storm are difficult to define, the 24-hour rationale can be utilized to approximate a year's mass emissions.

A third set of emission figures was developed using a computer analysis of the last 62 years of rainfall records from the Federal Office Building raingage. This approach assumed that overflows occured for rainfall exceeding 0.02

iph and the events were summarized in that fashion. There was an average of 82 events per year with a duration of 2½ hours and an equivalent rainfall of 0.17 inches.

An assumption was made that the City-wide runoff factor would be 0.65. The wet weather area utilized in this study was 24,014 acres. The total area consisted of three treatment plant drainage areas which contributed the following acreage and percentages to the total:

Plant	Sewered Area	A C	%AC
North Point	7,516 acres	5726	37%
Richmond-Sunset	9,004 acres	4999	32%
Southeast	7,494 acres	4712	31%
TOTAL	24,014 acres	15434	100%

Utilization of the percentage of the contributing wet weather area allows the distribution of the total mass emissions into the three drainage areas tributary to the treatment plants.

Plate IV-32 shows the comparisons on a raw mass basis and Plate IV-33 gives the same comparison on a percentage basis. All emission data shown thus is based upon raw emissions from the watersheds. The combined sewer overflow portion of the total emissions is calculated by multiplying the "due to storm" emissions by the percentage of runoff that overflows (the fraction of the total greater than 0.02 iph) and adding to that the dry weather flow contribution calculated as the dry weather emissions times the percentage of time that overflows occur.

In reference to combined sewage overflows, it is estimated that about 7% of the COD, 25% of the TSS, and 17% of the floatable emissions generated on an annual basis in the City are bypassed to the receiving waters.

The effects of land usage are reflected in the combined sewer overflow emissions in that with the existing system the dry weather flow contribution to the overflow emissions increases with increased intensity of land use as a result of increased dry weather emissions. The effect of the dry weather flow contribution is evidenced most obviously in the combined sewer overflow emissions of phosphates which are from about double to triple the emissions of the "due to storm" fraction of the total. The land use impact is most strongly reflected in the NPWPCP zone emissions where the combined sewer overflow portions of the total exceed the "due to storm" fraction for COD, TSS, TN, OPP and HEM, all as a result of the higher amounts of dry

weather emissions which occur as a product of increased land use intensity.

It is very significant to note that in the Richmond-Sunset and Southeast zones and in the City as a whole, the only constituent for which the combined sewer overflow fraction exceeds the "due to storm" fraction by an amount that can be deemed significant within the accuracy of the method is phosphates. For all other constituents, the combined sewer overflow contribution is less than or equal to the "due to storm" fraction of the total which is justification for the conclusion that the City's existing combined system provides greater control of annual emissions.

Per-Storm Basis

The approach taken to develop a comparison of the average daily mass discharges from the Baker Street Basin during dry weather days and average wet weather days was to average the dry weather flow and mass emission data shown in Plate IV-32 on a per-day basis, and to average the storm runoff flow and mass emissions data reported in the same set of tables on the basis of the number of overflow events estimated over the data period. With this approach, it was possible to develop the comparison of the average flows and mass emissions for a dry weather day and an average wet weather day as shown in Plate IV-34.

The data shown is based upon an average of 46 days of overflows in any given year with 206 hours of total overflow time. It is assumed that 68% of the runoff overflows the system.

The information reported in Plate IV-34 provides a basis for assessing the typical impact of wet weather conditions on the day-to-day wastewater emissions from San Francisco. The emission rates of both TSS and floatable materials show consistent increases from dry to wet weather conditions. Both the TSS and floatable materials have major aesthetic significance in terms of maintaining the beneficial uses of the receiving environment.

As noted previously in the annual discharge comparison, the effect of land use is noticeable when the NPWPCP and the RSWPCP zones are compared. It is also interesting to note that the overflow emissions are, with the exception of TSS and floatables and also HEM in the RSWPCP zone, about the equivalent of a primary effluent when compared to an average dry weather day.

The significance of these conditions will be discussed in the problem analysis section of this report.

Receiving Waters

All wastes emanating from the Sacramento and San Joaquin Valleys, the Delta area, and the two reaches of San Francisco Bay must pass through the Golden Gate. The water quality in this area reflects the cumulative effect of over 40,000 square miles of agricultural and storm drainage and the ultimate outlet for the wastes of a combined population of over 5 million people.

These waste discharges not settled or degraded enroute ultimately pass through the Golden Gate to the Gulf of the Farallones and disperse in the Pacific Ocean. This flushing of the bay is achieved through the advective flow of the Sacramento and San Joaquin Rivers and other minor tributaries and through the flushing due to the ebb and flow of the tide. Typical Delta outflows as they now exist are shown in Plate IV-35. For most of the year tidal flushing is the predominant mechanism for the flushing action that occurs and has the most influence on San Francisco's northern and western waterfronts.

Unfortunately many of the standards set for the protection of the Bay and Ocean are specified in subjective terms and no quantitative objectives are specified. This lack of quantitative criteria is due to two factors; first, the difficulty in relating various established parameters to various required degrees of protection, and secondly, a lack of definitive methods of quality measurement for many of the uses established.

However, there are various existing observations and measurements that have been taken which illustrate the degree of pollution existent during wet weather periods as compared to dry weather periods.

Foremost among the existing quantitative requirements is that for the bacteriological quality of the waters. Past records indicate that during much of the winter season the waters of the shoreline exceed the State public health standards for whole body water contact sports. This is aptly illustrated in Plates IV-36 and IV-37 . Estimates of the actual number of days that the waters are contaminated due to wet weather overflows vary, but a reasonable estimate is 171 days per year. However, as the existing requirements are now written, it is conceivable that beaches could be posted for as long as 8 months per year.

Due to the intermittent nature of wet weather overflows, the Public Health Departments posts the beaches from October to April every year. The continuous maintenance of waters that are bacteriologically "safe" for whole body water contact sports rates a high priority in the order of quality parameters to be maintained.

A second set of parameters readily observable to the lay-public involves the beneficial uses of the aesthetic enjoyment of the waters. Observations of degrees of degradation range from the turbidity fields resulting from combined sewer overflows, storm sewer discharges, and natural runoff to the detritus that washes up on the beaches. The deposited solids range in composition from natural materials such as leaves, kelp, and driftwood to styrofoam fragments, grease particles, garbage and fecal material. Monitoring for this set of materials consists of visual observations on the beach areas.

Three sampling programs (once during dry weather and twice during wet weather) were conducted at Outer Marina Beach in 1969-1970 to provide background data on:

(1) The physical, chemical, and biological characteristics of surface waters and the benthos; and surface water concentrations of coliform organisms and floatable particulates.

(2) The concentrations of oil and grease (HEM), and the particle size characteristics of sediments in the beach intertidal zone.

Information developed in the sampling programs was combined with information available from local, state, and Federal sources (including the Bay-Delta model of the United States Army Corps of Engineers) to provide a reference description of the study area.

The findings provide insight into the quality and hydrodynamic characteristics of the receiving waters in this area, the impact of dry and wet weather discharges on receiving water quality, and the correlation between receivng water and beach intertidal zone quality of Outer Marina Beach. Observations made on the Bay-Delta Model indicate that significantly different circulation patterns exist in the waters 500 ft or more offshore compared to the nearshore zone (where the outfall for the dissolved air flotation facility is situated). Beyond the 500 ft nearshore zone there is a general easterly movement of bay waters on the flood tide and westerly movement on the ebb tide. The above patterns also exist within the nearshore zone but the westerly water movement breaks down during ebb slackening into a counter-clockwise eddy and during the flood slackening into a clockwise eddy. The effects of these eddies, of which the former appears to be most significant, is to constrain the exchange of water in the nearshore zone with the main body of moving waters during much of the tidal cycle. This implies that eddy formation and dissipation play a significant role in delimiting the movement of bay water past the dispersion zone for the Baker Street Outfall. Additionally, the current patterns during ebb tide transport waters from the southern estuary northerly and westerly into and past the receiving waters contiguous to Outer Marina Beach, transporting simultaneously quantities of diluted treatment plant effluents, and during wet weather, combined and storm sewage flows into the area of Outer Marina Beach.

Receiving Water Quality Parameters

The most significant variations between dry and wet weather conditions of all water quality parameters considered in the above surveys were observed in the levels of the flotable material and coliform MPN. The coliform MPN parameter is an indicator organism associated with the presence of waste matter potentially of human origin. The floatable material parameter is an aesthetic parameter indicative of the presence of material floating on the surface of the receiving waters.

Baseline information on the coliform MPN parameter at Outer Marina Beach was developed from data available from the City's routine sampling program. Based on an analysis of this data, the median coliform MPN levels were found to vary significantly between dry and wet weather conditions at both shoreline and off-shore sampling stations. The median coliform MPN level at the Outer Marina Beach sampling station from mid-1966 to December 1968 was 320 MPN/100 ml in dry weather as compared with 1,960 MPN/100 ml on rainy days (days which by definition had greater than 0.02 inches rainfall). The coliform MPN level increased by a factor of six from dry to wet weather conditions. The median coliform MPN levels observed at sampling stations 250 to 1,500 ft offshore were 140 MPN/100 ml in dry weather and 1,000 MPN/100 ml in wet weather, i.e., were over seven times greater in wet weather than in dry weather. Thus, a similar level of change in the median coliform MPN level from dry and wet weather was observed in both the shoreline and offshore stations.

The tidal current stage was also found to be a significant variable affecting the wet weather coliform MPN level at the sampling sampling station at Outer Marina Beach. The median coliform MPN levels at flood and high slack stages during wet weather were approximately 1,200 MPN/100 ml which is one-third the median levels observed at this station for ebb and low slack stages. The distributions of coliform MPN levels between flood/high slack and ebb/low slack stages were found to be significantly different and demonstrates the impact of wet weather waste discharges easterly of the receiving waters contiguous to Outer Marina Beach on the quality of those waters.

Fluctuation in Coliform MPN Levels After Cessation of Wet Weather

The fluctuation of coliform MPN levels upon cessation of rainfall was evaluated for Stations 24 (Marina Pump Station Outfall) and 25 (Outer Marina Beach) by statistical analysis of the coliform MPN data developed in the City's sampling program. The results of the analyses indicate that the median coliform MPN level at Station No. 24 decreased from levels in excess of 2,500 MPN/100 ml on rainy days to the background dry weather average of 270 MPN/100 ml within five dry weather days. Similarly, the mean coliform MPN levels at Station No. 25 (Outer Marina Beach) decreased from levels in excess of 1,200 MPN/100 ml on the rain days to levels below the background dry weather average of 320 MPN/100 ml within five days. The consistency of these observations for the coliform parameter reflects the natural capacity of the bay system to attenuate and/or disperse pulse inputs of organisms of sewage origin to the system. It is not yet known if the rate of decay of coliform organisms to dry weather levels is also representative of the decay rates of other water quality parameters. However, it is apparent from the foregoing analysis that the number of days on which the quality of the receiving water will be impaired by wet weather emissions is a function of the discrete time intervals between storms causing combined sewage overflows as well as the time span between the initial and final overflows of the rainy season.

The foregoing analysis of coliform decay rates of the Outer Marina Beach provides an intial basis for estimating the number of days in a year during which the quality of the receiving environment will not be acceptable to sustain the beneficial uses designated for it.

Floatable Materials

Another area of concern with respect to receiving water quality is the nature and magnitude of floatable materials observed on the water surface under wet and dry weather conditions. The average floatable particulate concentration observed in the wet weather surveys was 10.5 mg/sq m, or over one order of magnitude greater than the average concentration of 1.5 mg/sq m observed during dry weather. The average dry weather concentration in surface waters at Outer Marina Beach was observed to be 0.44 mg/sq m westerly of the existing Baker Street Outfall alignment, and

0.68 mg/sq m easterly of this alignment. The wet weather levels were 7.8 mg/ sq m in the westerly sector and 12.1 mg/sq m in the easterly sector, indicating that floatable particulate levels in waters easterly of the alignment are over 50 percent greater in all seasons than they are in waters in the westerly sector. It is noteworthy that there are no sanitary sewage discharges or combined sewer overflows westerly from the Baker Street Outfall to Bakers Beach outside the Golden Gate. The HEM fraction of the floatable particulate concentration increased from 0.01% in dry weather to two percent in wet weather. Because of the order of magnitude variation of floatable particulate levels between dry and wet weather, it is evident that the HEM levels varied by three to four orders of magnitude between dry and wet weather in the receiving waters of Outer Marina Beach. In spite of the significant increase, most (greater than 98%) of the particulate floatables found were of plant or animal rather than of sewage origin.

Consequently, upstream treated and untreated waste discharges have a significant impact on floatables levels and the oil and grease fractions thereof found in the receiving waters of Outer

Marina Beach. The increased levels of floatables in receiving waters under wet weather conditions corresponded to a factor of two increase in the HEM content of the surface sand in the beach intertidal zone, as discussed below.

Other Water Quality Parameters

During the receiving water surveys of 1969-1970, three sets of Secchi Disc readings were made in the receiving waters to measure the depths of light penetration by this index. The Secchi Disc measurements varied from 2-1/2 to 3 feet during the first survey (February 1969) from 5.5 to 6 ft during the second survey (May 1969) and were three feet during the third survey (January 1970). It is evident from a comparison of these data with data obtained in the 1960-1964 period that the waters in 1969-1970 were less turbid than was indicated by the average conditions prevalent in the total bay system in 1960-1964 (as reported by the University of California- "A Comprehensive Study of San Francisco Bay.")

The concentration of microplanktonic organisms in the area offshore from Outer Marina Beach varied from 64,000 to 92,000 organisms per liter of water sample. These values were somewhat lower than would be anticipated on the basis of information developed in previous studies and additional data are being obtained to confirm these observations during 1971.

Benthos

Measurement of the HEM, total nitrogen, and total sulfide contents of sediments and of biota present in the sediments of the bay offshore from Outer Marina Beach were conducted at five stations in the nearshore zone. The sediment samples were found to contain primarily sand with 9 out of the 10 samples containing at least 70% sand on a weight basis. The concentrations of HEM in sediment samples varied from 55 to 379 mg/kg of sediment, with an average value of 212 mg/kg, or nearly 50% less than the average level reported for the Central Bay area for the 1960 64 period. The total sulfide values in the sediment samples at the five sampling stations varied from a trace to 0.0376 mg/kg. These observations on sulfide supported the concept advanced by the University of California (1960-1964) that the highest total sulfide contents in San Francisco Bay are associated with the samples that have the least concentration of sand.

The total nitrogen levels of the benthos samples at the sampling stations varied from 0.07 to 0.72 mg/g of dry sediment. These levels are significantly less than the concentrations of 3 to 4 mg/g reported during the 1960-1964 period. Thus, as a general trand, the results of the 1969-1970 surveys indicated significant reductions in basic benthic parameters relative to observations made in the 1960-1964 period.

Beach Intertidal Zone

The beach intertidal zone survey at Outer Marina Beach indicated that a positive trend existed in changes in HEM levels in the receiving waters and beach intertidal zone between dry and wet weather conditions. The mean specific HEM content of intertidal zone sand varied from 20 mg/kg in dry weather to 97 mg/kg during the April 1969 wet weather survey and 62 mg/kg in the January 1970 wet weather survey. These changes correlate positively with increases in HEM levels observed in the floatable particulates in the receiving waters, which were from three to four orders of magnitude (as discussed above), and the changes offer strong evidence of the interdependence of receiving water and beach intertidal water quality in the study area.

Implications of Receiving Water Evaluation

The foremost findings of the receiving water evaluation are that there exist direct interrelationships at Outer Marina Beach between:

(1) Receiving water and beach intertidal zone quality.

(2) Receiving water quality in tidal current stage, with a deterioration in water quality occurring on ebb and low slack tidal current stages.

(3) Receiving water quality and the climate, with a marked deterioration in water quality observed at points in the Central Bay occurring during wet weather. It is possible that this deterioration extends to the whole bay during these periods.

(4) Wet weather coliform MPN levels in the receiving waters recede to the background dry weather level after an elapsed timed period of five dry weather days following cessation of rainfall.

CHAPTER V

PROBLEM ANALYSIS AND POSSIBLE SOLUTIONS

As was stated in Chapter I, the objective of this report is to develop a City-Wide Sewerage Master Plan for the control of combined sewer overflows as necessary to protect, preserve, and enhance the waters of San Francisco Bay and the Pacific Ocean for all the designated beneficial uses which might be otherwise impaired as a result of combined sewer overflows from the City.

The traditional approach has been to devise immediate and separate solutions to localized problems of flooding and water pollution control. However, a systems approach which encompasses the mass balance relationships between system inputs and outputs provides the only rational method of assessing the problem. A series of systems are involved, the combination of which constitutes the total bay system of 40,000 square miles. Wastewater flows from San Francisco in the amount of 36.5 billion gallons per year from domestic and industrial sources and 8.8 billion gallons of storm runoff constitute the annual water mass to be controlled prior to discharge. Of this total 39.3 billion gallons are now subject to treatment and 6 billion gallons overflow directly without treatment. 28 billion gallons of domestic and industrial wastes are discharged to the Bay together with 4 billion gallons of overflows annually. 11.3 billion gallons of treated wastes and 2 billion gallons of overflow are discharged directly to the ocean.

Projected flows are shown on Plate V-1 for a fifty-year period.

San Francisco receives 20.33 inches of rainfall average annually as based upon 62 years of records at the USWB Federal Office Building gage. On the basis of the 24,500 acres of sewer service area in the City and a runoff coefficient of 65%, there are 8.8 billion gallons of runoff derived from the 20.33 inches of rainfall. Of this amount about 2/3 drains to the bay and 1/3 drains to the ocean. The existing system diverts about 2.8 billion gallons of the total 8.8 billion gallons to treatment. The remaining 6 billion gallons overflows with about 4 billion gallons going to the bay and the remaining 2 billion gallons discharging to the ocean.

The total rates of runoff vary from 100 MGD at 0.01 ph of rainfall to 24,200 MGD for a 100 year storm.

On an annual basis there is adequate treatment capacity to provide treatment for all runoff that passes through the system. In fact, on an annual basis, there is hydraulic capacity for 124 billion gallons of flow. This represents the capacity for up to 200 inches of rainfall above the existing domestic flow if the rainfall were evenly distributed over the full year. To provide treatment at a maximum overflow rate of 2500 gal/Ft^2 - day would require 55 MCF to accommodate a 5 year rate, 66 MCF for a 10-year rate, 77 MCF to accommodate a 25-year rate, 83 MCF to accommodate a 50 year rate, and 97 MCF to accomodate a 100 year rate. Such volumes would function as storage basins up to the time that the tankage became full, after which the treatment operation would be initiated. Thus the provision of adequate treatment capacity to handle high flow rates also provides large storage volumes. The alternative to providing such large treatment capacities that would rarely be used, is to consider the use of storage to retain the excessive flow for treatment through intermediate capacity plants when the available capacity exceeds the runoff.

Hydrology

The only reliable and detailed long-term record of precipitation occurrence in San Francisco is that of the U.S. Weather Bureau for one gaging station. Sixty-two years of this record (1906-1968) were analyzed to determine the dimensions of the phenomenon. Records from the Richmond-Sunset

Water Pollution Control Plant gage were also given similar analysis. The record length is only 20 years and is limited in that sections are omitted.

Two rainfall characteristics that are taken from the rainfall record data for the purpose of sizing specific elements of the sewerage system are: (2) rainfall intensity (which sets treatment and conduit rates), and (b) rainfall volume (which sets storage volumes required).

Because of the paucity of data available, previous designs assumed that the U.S.Weather Bureau rainfall record was generally applicable throughout the City, and that the occurrence of particular intensities of rainfall was simultaneous all over the City. Over the 20 year period available the Richmond-Sunset gage averaged two percent more annual rainfall than the Federal Office Building gage, hardly a significant difference. However, the Richmond-Sunset gage recorded from 13% less to 19% more rainfall than the Federal Office Building gage on specific years which indicates the variability on a yearly basis.

About 90% of the total rainfall occurs at rates equal to or less than 0.28 inches per hour. 98% of the rainfall occurs at rates equal to or less than 0.50 inches per hour.

90% of the time the rainfall is less than 0.14 inches per hour and that there is only about 1 hour of rainfall at rates greater than 0.45 inches per hour in the average year.

Very little work had previously been done on the San Francisco records in the analysis of the volumetric characteristic of rainfall prior to this effort. A computer program was developed to synthesize varying storage volumes together with varying rates of treatment to analyze the effect of combinations upon overflows. For definition purposes an overflow event begins when the rate of rainfall exceeds the rate of treatment and either: (1) overflows directly to the receiving waters, or (2) goes into storage when available. The event ends when the withdrawal from storage exceeds the input or, in the case where no storage is provided, when the overflow is ended.

With this definition, the 62-year rainfall record from the Federal Office Building gage and the limited record at the Richmond-Sunset gage were analyzed. The results of the analysis are presented in Plate V-2 , which illustrates the relationships between given combinations of storage and treatment with overflow frequencies and overflow volumes.

The Federal Office Building gage indicates no significant differences from the Richmond-Sunset gage in the ranges of the composite presentation shown. The data does diverge a little for overflow events of infrequent magnitude. For the purposes of this report the longer record length is used for component sizing.

The records available from these gages provide the essential factors for the evaluation of the efficiency of any combination of storage and treatment in reducing the degradation to the receiving waters through reduction of overflow volumes and frequency. The factors for evaluation are the quantity and rates of combined wet and dry weather flow which are routed through treatment, together with the efficiency of the treatment processes, the quantity of untreated overflow, and the frequency of overflow occurrence.

Use of the above described program and the Federal Office Bldg. record with 0 storage and at treatment rate of 0.02 inches per hour as shown in Plate V-4 provides the baseline or existing condition data. By this method it has been determined that approximately one-third of the runoff is treated and discharged by the water pollution control plants and that the other two-thirds, or about 6.0 billion gallons of runoff, overflows without treatment. This volume of overflow occurs during an average of 206 hours per year. On the average there are 46 days in the year during which 82 overflows occur. The estimated pollutant emissions due to these overflows has been described in Chapter IV.

The storage to contain all overflows from the greatest recorded storm utilizing the existing treatment rates would be 240 million cubic feet. This storage volume is then the upper limit of an all-storage scheme and exceeds by a factor of 2 the volume requirement of an all-treatment scheme.

The data presented above is necessarily based on two assumptions: that the runoff loss is 35% and that rainfall occurrence is uniform over the City. Each is a significant parameter is determining the total volumes of runoff. No verified data exists on the losses experienced in the rainfall-runoff process, although some measurements were made in the recent characterization study, and more data will be forthcoming during the succeeding wet weather seasons.

Storms were monitored during the 1969/1970 rainy season by a system of 19 gages as shown on Plate V-5A yielding results which show a 15% lower overall average volume of rainfall over the whole City than that indicated by the gauge at the Federal Office Building. Since the time correlation for the 19 rain gauges which were operational during that season was poor, the above percentage indicates only the result of spatial acquisition. Rainfall data collection during the 1970-71 year via the San Francisco Hydraulic Hydrologic Data Acquisition and Recording system and the analysis of this data through a plotting routine, SYMAP has provided a more graphic illustration of the spatial and temporal differences. Plate V-5 illustrates one storm of high intensity showing significant variation. Low intensity storms observed to date have not shown dramatic patterns.

The spatial and temporal differences observed in the occurrence of rainfall leads to the conclusion that a system of interconnection would result in more efficient utilization of facilities; the use of real-time computer-actuated control, based on sensing the direction and the likely volumes of rainfall, with a constant concurrent updating of the status of the system, would permit the use of all capacity throughout the system. The result would be the construction of fewer and smaller facilities which would be used for the overall system rather than only for discrete segments of the system. However, an extended period of data acquisition must precede the development of "decision tables" to be implanted in the controlling device.

Other characteristics of the various combinations of storage and treatment include the number of events that would have occurred, the volume of overflows, the duration of overflows, and the number of days of overflows. With the previous assumptions, this data can be used to evaluate any control system with regard to possible overflow quality and mass emissions of constituents.

This information can also be related to the sewer system and to the effect of any control system upon the transport portion of that system.

The sizing of the systems presented in this report is based upon the records available from the FOB gage. This data represents the best information available at this time and all other data indicates that any design based upon this gage will likely be conservative with regard to sizes and costs. Refinement of the design will take place over the next five years as more data becomes available through the City's collection system.

Plate V-7 represents a composite of the effects of various combinations of storage and treatment with regard to the frequency of uncontrolled overflow occurrence. It is apparent that, given a desired frequency of overflow occurrence, increasing treatment decreases the storage requirements and that, for any given storage volume, increasing treatment capacity results in a lower occurrence frequency. Further inspection of the plate also indicates that the law of diminishing returns results in increasingly greater storage requirements for any treatment rate to attain the lower occurrence frequencies of overflows. Through the application of cost factors for storage and treatment facilities, optimum design points for minimum cost and for levels of control can be derived. In the analysis presented later in this chapter no recognition is given to the effect of the storage provided by treatment facilities during the inception of an event. This provides an additional factor of safety with regard to size selection for overflow recurrence interval which must be given further evaluation together with the temporal and spatial rainfall effects.

Sewer System

The present design criteria is the conveyance of a 5-year intensity storm without flooding. When rainfall intensity exceeds the design rate, surface transport and flooding occurs. As can be seen on Plate V-8 , there are numerous locations in the City where surface waters will accumulate until capacity in the system will accept them. If retention basins were located at these sites with appropriate street drains, surface waters would flow to the basins. Such facilities installed in these locations should provide for greater public protection from the inconvenience of surface ponding.

Another benefit of retention basins upon the conveyance system is through limiting the flow to the downstream conduit conveyance capacity.

The location of stormwater retention or storage basins in any particular sewer system has a beneficial effect on the available downstream conduit transport capacity in terms of historical rates of rainfall. The selection of storage basin locations within any stormwater conveyance system can be made such that the main trunk sewers of the system downstream of the basins will be upgraded with regard to the size of the storm that may be conveyed before exceeding the sewer capacity. The volume of storage facilities considered must adhere to the following criteria:

(a) The storage volumes utilized shall not be less than 1/2 million cubic feet; this restriction stems from the consideration of economics of construction of such basins.

(b) The required volume for storage is equivalent to the volume of runoff derived from 1" of rainfall on the contributing fraction of the watershed which is not already tributary to a basin. This is based upon the frequency of overflow of such basins which historically would occur when used in conjunction with the next criterion.

The evacuation of flow out of storage is continuous and is equivalent to the rate of runoff from a steady state rate of rainfall of 0.10 in/hr from all upstream tributary area. This criterion, in conjunction with (b) above, provides a storage volume which historically will overflow only one time every five years.

For sewers in the City larger than 3 ft. it is estimated that the cost to replace all sewers with inadequate capacity in this size range is $ 77 million. The cost of basins will be offset to some degree by the equivalent costs of installing additional or longer sewers in those areas where inadequacies now exist. All conceptual design costs will be evaluated with regard to this aspect. In the Selby sewer system for example, there are $21.74 million worth of inadequacies. The installation of the basins as shown in the example would reduce the costs of replacing these inadequacies by $ 30 million of the $77 million. This indicates a total inadequacy of all sewers of $150 million, about $50 million of which could be saved by means of the retention system.

A further benefit of retention basins is in the potential for flushing the conveyance system by storage flows during portions of the day and subsequent release. This may reduce maintenance costs to the lower portions of the system in the subsidence areas.

Receiving Waters

Ultimate consideration for waste disposal must be directed toward the capacity of the receiving waters to assimilate any discharges. Given the criteria for discharge and having quantification of the raw wastes to be treated leads directly to the development of the required treatment. The analysis requires the knowledge of the quality of the flows to be handled. Up to this point consideration has been given only to hydraulic balances. Similar balances must be made for the waste constituents to complete the picture.

Quality Mass Balance

Existing condition mass balances of constituents for the City were presented in Chapter IV. The total of these masses will not be changed by any control system but the output through the various distribution and treatment facilities will change as decisions are implemented. Projected masses will change however, as land use characteristics are modified in the future. As a first consideration then land use projections must be evaluated.

Land Usage

Each water pollution control plant has a tributary area which consists of distinct subareas or watersheds which, under dry weather conditions, are the source of sanitary flow to each of the treatment plants. Under wet weather conditions these watersheds also contribute surface drainage to the sewerage system and as previously discussed, various fractions of the wet weather flows are transported to the treatment plants with the remainder overflowing to the Bay and Ocean.

Various sources of land use and census data are available including future projections on the basis of the City's census tracts. Census data is collected every ten years and includes land use by type, housing units and residential population. This information is maintained by the Department of City Planning. Other sources of information are available in the Bay Area Simulation Studies (BASS III and IV) completed under the San Francisco Bay Delta Water Quality Control Program. This information source includes employment,land use, residential population and housing densities projected for each census unit in the Bay Area.

This data has been collated for the City and the base-line data developed for each watershed within the system. (Plates V-9,V-10, V-11 and V-12). The projected conditions are for use with the City's sewerage system model in evaluating the effects of changes in land use and population based upon the Master Plan to be adopted.

This data also serves as the basis for projecting dry weather conditions. Flow projections as shown in Plate V-13 were derived from projected city population.

As can be noted from the projections no drastic changes are anticipated within the City. Trends indicate a decrease in single family residences with increases in multi-family residences. Population will increase slightly. A significant increase in employment is projected. The impact of this will be noticed through increased peak flows and increased service industry such as restaurants.

Based upon these projections it is unlikely that there will be any significant changes to the mass emissions beyond increases consistent with the flow projections. All of this data will be programmed into a sewerage system model and effects noted in the detailed system design.

Runoff Quality

Runoff emissions were derived in Chapter IV and were based upon coefficients related to inches of runoff from the existing surface system. There are few controllable factors which might modify these values. Obviously, if the imperviousness of an area increases the runoff will increase. Qualitatively, it is doubtful if this factor will modify the emissions to any degree predictable by present methods. Street cleaning methods will influence the runoff quality but no predictions of the effect of increased efficiency are possible without further study. Efforts are in progress at this time to further evaluate this aspect of the runoff problem. Again it is doubtful if changes in the foreseeable future will modify the existing runoff quantification to any significant degree within the accuracy of the method of quantification used.

A further factor which influences the quality of combined sewer flow in the use of catchbasins in the storm runoff collection system. As noted in Chapter IV, there are approximately 25,000 catchbasins located within the system. Each of these basins has the basic configuration shown in Plate V-14 and has a storage volume of about 24 ft^3. Approximately 50% of the total number of catchbasins in the City are reported to be cleaned annually by the City's maintenance and operations forces. Cleaning is accomplished with an eductor truck. Thus, a first assumption that can be made is that any given time the average contents of any catchbasin amounts to about 12 ft^3.

It was observed that there is an apparent relationship between the time from the start of rainfall and the changes of quality observed in the combined sewage flow. At first it was thought that this phenomena could be characterized on temporal terms relative to the time after the onset of rainfall(1). Subsequent observations have indicated that this is not the case but that the quality perturbations vary with the flow rate indicating the influence of scouring and transport within the total system(2). This would include street, gutter, roof, and yard runoff, pipe scouring and transport, and catchbasin flushing.

(1)ESI-C&CSF - Characterization of Combined Sewer Overflows, 1968.
(2)ESI-C&CSF - Pre-eval. Studies.

Catchbasins are probably functioning as accumulators of various materials which are disgorged during periods of heavy rainfall. If the catchbasins functioned as mixing basins, the disgorging would occur over a period of time with a curve of concentration of flow similar to that shown in Plate V-15 . While the curves shown are for mixing basins under ideal conditions it is surprising how close the curve shapes for the multiple units approaches the concentration profiles for combined sewer flow.

Given this intuitive set of observations, it was determined that the catchbasins should be removed from the system to eliminate this sudden release of mass emissions.

Arguments against retaining the use of catchbasins include the above described pollution potential, the high cost of maintaining the catchbasins (cleaning), the flooding created by clogged basins, and the feeling that the basins do not function as odor seals and vermin traps in any event. To verify this last premise the survey was conducted which was discussed on page IV-6.

Based upon these observations three tentative conclusions may be put forth:

1. There is little relationship between odors in the

sewers and odors observed at catchbasins;

2. Catchbasins as presently maintained do not function as sealing mechanisms except randomly; and

3. Significant amounts of debris are stored in catchbasins.

Within each of 7 areas of the City, 9 catchbasins were sampled at approximately weekly intervals. A composite of sample of the catchbasins from each area was made and chemical analyses run on these samples. Overall the material sampled has a strength similar to an industrial waste. COD's were particularly high as compared to the BOD's which were on the average about equivalent to sanitary sewage. Grease analyses yielded very high values. Total suspended solids were high as might be expected but the volatile suspended solids were also high. Total nitrogen results varied but averaged what would be expected from a weak sewage. Total phosphorus results were consistently low. Given these constituent values it is apparent that catchbasins constitute a potential odor source (through putrefaction of organic solids) by themselves.

Significant reductions in "due to storm" contributions of TSS, TN and TP are not expected to result from the elimination of catchbasins, however, reductions in COD and grease may be significant.

It is reasonable to conclude at this time that catchbasins do not perform the function for which they are currently justified, i.e. odor and vermin sealing of the sewer system.

Evaluation of Control Systems

For some constituents the existing system provides a degree of control equal to or better than separation would provide. The "due to storm" fraction which represents surface runoff, catchbasin flushings, and sewer system scour, appears to be the equivalent of the separate storm sewer discharge fraction of the total mass emissions from the City. Plate V-16 shows comparative concentration values, for separate urban surface runoff and the derived "due to storm" runoff for San Francisco.

Obviously, there is one significant difference between urban surface runoff and combined sewer overflows and that is coliform organism concentrations. Monitoring of dry weather flows, combined flows and flows in separated portions of San Francisco sewers has resulted in the following mean values of coliform MPN.

Dry Weather Flow (Raw): mean - 29 $\times 10^6$ MPN/100 ml

Combined Sewage: mean 6.2×10^6 MPN/100 ml

Surface Runoff (Separate System): 8.2×10^5 MPN/100 ml

Thus, there is an order of magnitude difference. The effect of coliform discharge is directly related to receiving water quality and will be evaluated later in this chapter.

On a daily basis a different comparison becomes apparent. Using the total City as a basin, the mass of wet weather TSS and flotables exceeds the average dry weather plant emissions by 2 to 5 times. The nitrogens and phosphates are 1/4 to 1/2 of the average dry weather emissions and the HEM is about the equivalent of the dry weather emissions. The higher levels of flotables and solids discharges relate to the observed buildup of these materials on the shoreline and the aesthetic degradation associated with the overflows. It does not, however, account for the differences of one to two orders of magnitude observed in terms of flotables and grease both on the shoreline and in the receiving waters. Other discharges must be implicated to account for the total increase observed.

Receiving Waters

The quality of the receiving waters of San Francisco Bay are influenced by numerous municipal, industrial and natural discharges of waters of varying degrees of quality to the Bay-Delta system. Due to San Francisco's location at the mouth of the Bay, the waters contiguous to the City are affected by all upstream discharges.

As a first approximation at estimating the relative loadings on the Bay, the information available from the Bay-Delta study by Kaiser Engineers, published in June 1969 was used.

The Bay-Delta study reported pounds of pollutants being generated from the following sources:

1. Municipal and Industrial (M & I)
2. Natural Runoff (N R O)
3. Agricultural (Agric.)

It is to be emphasized that the information in Plate V-19 is the total pounds of raw pollutants emanating from the various areas and that the pollutants reported are those prior to treatment, if any. Study Areas A & B are shown in Plate V-17 and Water Quality Zones are shown in Plate V-18. The year 1965 was chosen because the data presented for this time period was more readily available in a published form than was comprehensive data for any other year. The difference between 1965 and 1970 was also considered to be negligible for the purposes of this report.

The table of raw pollutants generated in Areas A and B as set forth in the Bay-Delta Report has been updated by the following information to create Plate V-19:

1. Records from the Southeast and North Point Water

Pollution Control Plants for BOD, Flow, Grease and Oil, and Total Suspended Solids for the time period of July 1969 to June 1970.

2. Data extrapolated from a week of testing conducted at the North Point and Southeast Water Pollution Control Plant for Nitrogen and Phosphorus.

3. Mass emissions extrapolated from a report on Pre-Construction Studies of the Dissolved Air Flotation Process at Baker St., for the rainfall year of July 1969 to June 1970.

Considering the total possible discharge to the Bay insofar as TN, TP and TSS are concerned, the overflow from the City represents a very small fraction of the total. Total removal of TN for example would reduce the natural runoff and agricultural contribution by only 2%. Similarly, total removal of TSS would reduce the natural runoff mass by 0.9%. The comparisons are of similar magnitude on a water quality zone basis. For example, oil and grease from overflows accounts for 28% of the total natural runoff contribution of zones 3 and 4, but only 2% of the total possible emission from zones 3 and 4. With regard to flotables, the emissions from San Francisco's overflows in zone 4 exceed the permissible total zone loading reported by Bay-Delta. However, the runoff fraction from agencies with separate systems exceeds the limits also.

Further insight into the problem of flotable material is afforded through the U.S. Army Corps of Engineer's efforts in removing large debris for navigational purposes. About four times the amount of debris is collected during the winter as opposed to summer period from the bay system.

Given this relative impact of wet weather overflows, some conclusions can be drawn regarding control of emissions:

1. Restriction and control of wet weather overflows with regard to biostimulants as measured by nitrogen and phosphorus discharge will not provide any significant reductions over amounts discharged from all sources. Control of biostimulants should be directed toward dry weather municipal and industrial discharges and agricultural drainage as necessary to control undesirable aquatic growths. Any localized problems attributable to wet weather may be alleviated by discharge relocation and attainment of better mixing and dispersion.

2. Control of solids discharged during wet weather should be directed toward the removal of settleable solids and floatable materials. The total suspended solids loading of wet weather discharge from the City represents a small fraction of the total wet weather solids loading on the bay resultant from natural runoff. Settleable solids should be removed to control bottom deposits with the accompanying adverse benthic response

and floatables controlled to minimize aesthetic degradation of the waters and shoreline, and potential adverse health and ecological effects.

3. Control of BOD emissions during wet weather is not deemed necessary due to the lack of any problem encountered in maintaining dissolved oxygen levels under present conditions. Any increased removal will probably not result in any measurable change to the background levels of the receiving waters.

4. Control of the turbidity of wet weather discharges is not a major design parameter. As the regulatory agencies determine the appropriate levels it must be remembered that the natural runoff to the bay and ocean results in significant turbidity fields and that similar conditions would be existent regardless of whether the City's conveyance system was combined or separate.

5. pH control of wet weather discharges should be measured in terms of deviation from natural background conditions rather than the effluent limitation of 7.0 to 8.5 as is now RWQCB policy. Wet weather flows, in a separate system in the City when monitored, had pH's ranging from 6.1 to 6.8. Combined flows have slightly higher pH's but may still be below the 7.0 limitation. If separate system storm flow and natural runoff exceeds the limits then more stringent effluent requirements for combined flow seems unjustified.

6. Toxicity control will be attained through two avenues; first, attainment of the discharge criteria presented in Chapter IV and, second, source control for persistant toxicants such as heavy metals and pesticides derived from the sanitary waste flow component of combined flow. Successful attainment of these goals should result in compliance with all regulatory requirements and policy regarding discharge toxicity. Specific effluent limitations for specific constituents will have to be evaluated on a case-by-case basis as more data becomes available. It should be noted that in any case the discharge of discrete toxicants during wet weather via wet weather flows will not exceed about 5% of the dry weather influent mass. Thus, control of mass emissions to maintain safe receiving water residual levels is most effectively directed at the dry weather emissions as opposed to wet weather emissions.

7. The control of the bacteriological quality of wet weather discharges is governed by several factors. It can be stated that the objective of any disinfection facility is to prevent the degradation of the receiving waters relative to various beneficial uses and bacteriological limits resulting from the waste discharge. This can from the discharger's perspective be accomplished in one of two basic ways; either by

meeting the required quality limits in the receiving waters, or by meeting the limits directly in the effluent. The background levels of the Bay rise during the winter season to levels that are close to or exceed present requirements.

Thus to summarize with regard to wet weather flows and required treatment, the necessary treatment consists of eventually complete removal of settleable solids and floatables, chlorination to meet requirements either in the receiving waters or in the effluent. In attaining these levels the emission of other constituents will be reduced to levels at least equivalent to primary effluent. When discharged to the receiving waters through outfalls the effluent and receiving waters should meet all goals and requirements of the RWQCB for wet weather flows. Turbidity and discoloration requirements and possibly various specific effluent constituent limitations might be applied by the RWQCB so as to require adjustments to chemical treatment prior to sedimentation. Based upon the relative mass emissions for dry weather, wet weather and natural runoff loadings, more stringent requirements for wet weather overflow treatment would not result in any significant improvement or benefit to the receiving waters.

The foregoing discussion applies to the portion of the wet weather flow which is captured or controlled by the storage-treatment system. However, there is a portion of the annual flow during peak rainfall which overflows directly to the receiving waters. The amount which is lost is a direct function of the size of the storage facilities available. Plate V-21 illustrates the percent control relative to the runoff mass. Plates V-22 through V-23 illustrate the effect on mass emissions. Each succeeding reduction in number of overflows results in reduced emissions. In terms of immediate impact to the receiving waters, there does not appear to be any benefit in reducing the overflow frequency to less than once per year as the masses of the overflows are about equivalent for the discrete event. As the annual wet weather emission masses for the total City are about equivalent to one day dry weather discharge of primary effluent it does not seem reasonable to attribute any long term water quality degradation to overflows of about once per year.

The only water quality parameter of significance that will be measurably affected by overflows for a time after the occurrence is the bacteriological quality. This may, in fact, be the only water quality parameter aside from aesthetic considerations which is beneficially affected through providing facilities to reduce overflow frequencies to less than one per year.

There are two bacteriological standards now in use for water contact sports in addition to the maximum MPN of 10,000:

1) Not more than 20% of the samples in any 30 day period may exceed an MPN of 1,000 organisms per 100 ml.

2) The median MPN of any five consecutive samples must be less than 240 organisms per 100 ml.

These two requirements are essentially equivalent.

As presented in Chapter IV studies on the City's northern shoreline have indicated that it takes about five days following an overflow for the receiving water coliform levels to recede to the 240 org/100 ml level. Allowing another five days to develop a median of 240 results in 10 days of violations per day of overflow. At present the rainfall events overlap to produce longer periods that average much less than 10 days per event.

To provide a better data basis for analysis the derived days of violations were compared to the number of days of overflow occurrences. This resulted in the curve shown in Plate V-27 which can be used to estimate days of violations given the days of overflow occurrences in a year. From this the effectiveness of any system can be related to the bacteriological quality of the receiving waters attributable to overflows.

Treatment Facilities

As presented above, the existing system treats about 30% of the wet weather flow from the City at this time in the three existing treatment plants. The apparent optimum combination of treatment and storage will entail increasing the treatment capacity to a maximum rate capacity of about 1000 MGD during wet weather from the present 340 MGD.

At this time the North Point and Southeast Plants are operating under cease and desist orders because of violation of the existing RWQCB requirements. The Richmond-Sunset plant, while not under cease and desist order is also in violation of requirements.

North Point Water Pollution Control Plant.

The major deficiencies at the North Point plant include inefficient solids removal and improper effluent disposal.

Solids removal efficiencies are affected by the large influent pumping sump and the long detention time of sludge in the sedimentation tanks. Both of these conditions cause septicity in the sludge with subsequent rising of sludge to the surface and carryover of solids with the effluent. Modifications of these facilities to minimize these effects, together with improvement of the scum and sludge removal systems, will largely alleviate this problem. Further chemical treatment facilities will be added to further reduce the discharge of solids and various constituents amendable to chemical removal. Additionally, to insure continuous reliability in sludge disposal, a second sludge force main to the Southeast plant should be installed.

Improvements to the effluent disposal system will involve construction of a deep water outfall. The outfall extension will consist of approximately 1800 to 2000 feet of onshore pipeline and 4800 feet of submarine pipeline. The submarine pipeline will include 1850 feet of diffuser section located about 70 feet below mean lower low water. Design of the outfall will be such that dilution in excess of 100 to 1 will be achieved at all flows up to 350 MGD.

With these improvements, all present quantitative requirements of the Regional Water Quality Control Board should be met. Compliance with qualitative requirements will depend upon the interpretation of the language of the requirement in question.

Richmond-Sunset Water Pollution Control Plant.

The primary deficiency at the Richmond-Sunset plant is the method of effluent disposal. In addition, sludge and scum removal methods from the primary sedimentation tanks and return of solids from the sludge treatment process cause higher than acceptable settleable solids in the effluent. A further problem is the apparent high toxicity of the influent and effluent.

Revision of the existing outfall system will include construction of an effluent pumping station and a submarine outfall extending approximately 6500 feet offshore. The effluent pumping station will have a capacity to pump 70 mgd through the submarine outfall against the highest high tide. The submarine pipeline will include 500 feet of diffuser section located about 58 feet below mean lower low water. Design of the outfall will be such that dilutions in excess of 100 to 1 will be achieved at all flows up to 70 mgd.

Improvements to the solids system will include means to pump sludge directly from the sedimentation tanks to the digesters thus eliminating the thickening tank and providing for chemical conditioning of digested sludge prior to dewatering thus eliminating elutriation tanks. These modifications will minimize the quantity of solids returned to the sewage flow and will result in lower settleable solids content in the effluent. Additionally, chemical addition facilities are planned to control solids and other constituents that can be chemically precipitated.

With these improvements and improved source control for toxicants, all present quantititative requirements of the Regional Board should be met.

Southeast Plant

Excessive return of solids from the sludge treatment process to the sewage treatment process seriously affects treatment efficiency at the Southeast plant. Major modifications must be made to the solids handling system at this plant to reduce this return. These improvements include (1) piping modifications to permit pumping solids directly from the sedimentation tanks to the digesters, (2) chlorination of North Point sludge prior to pumping it through the force main, (3) improvement of the thickening process for North Point sludge, (4) activation of all digesters to maintain low loading and to provide storage capability and operational flexibility, (5) elimination of the elutriation system, and (6) modification of the sludge filtering system.

To achieve the maximum initial dilution possible through the existing outfall the effluent pumping station should be operated so that effluent can be prediluted with sea water prior to discharge to the outfall.

Further reductions will be made through the installation of chemical treatment facilities.

With these improvements, all present quantitative requirements of the Regional Water Quality Control Board should be met.

At the completion of the 1973 Program there may remain modifications to comply with those requirements noted as compliance status unknown. Various levels of improvements will be required at each plant to accomplish this end. The following improvements are associated with the indicated goal levels.

North Point Water Pollution Control Plant

The North Point Plant with the greatest hydraulic loading and the least room for expansion presents the biggest challenge to the development of reasonably alternative treatment schemes. Many alternative schemes were discussed and studied with the result that the following seven were selected for economic analysis:

Alternative	Description
N1	Existing plant improvements (including North Point's share of solids handling improvements and additional thickeners at the Southeast plant) and the outfall extension.

N2 Existing plant and outfall improvements (including North Point's share of solids handling improvements and installation of additional thickeners and heat conditioning facilities at the Southeast plant) plus chemical treatment with low dose ferric chloride (15-45 mg/l), polymer (0.25 mg/l) for 12 hours per day and salt water (1200-1500 mg/l NaCl). Ferric, polymer, and salt water addition will be halted during periods of PWWF.

N3A Existing plant and outfall improvements (including North Point's share of solids handling improvements and installation of additional thickeners and filters at the Southeast plant) plus chemical treatment with low dose slaked lime (150-175 mg/l $Ca(OH)_2$), and recarbonation.

N3B Existing plant and outfall improvements (including installation of thickeners, filters, and incinerators at the Southeast plant) plus chemical treatment with low dose slaked lime (150-175 mg/l) and recarbonation.

N4A Existing plant and outfall improvements (including installation of thickeners and incinerators at the Southeast plant) plus chemical treatment with

N4B high dose slaked lime (300-350 mg/l) and recarbonation. Existing plant and outfall improvements (including North Point's share of solids handling improvements and installation of additional thickeners and heat conditioning at the Southeast plant) plus chemical treatment with high dose ferric chloride (100-150 mg/l) polymer (0.50 mg/l), and salt water (1200-1500 mg/l NaCl) with filtration and effluent pumping.

N5 Improvements under N4A plus filtration, breakpoint chlorination and carbon adsorption for the removal of nitrogen and organic compounds.

All alternatives were studied on the basis of ADWF of 71 MGD, PWWF of 200 MGD, and total suspended solids of 194 mg/l.

ADWF = Average Dry Weather Flow.

PWWF = Peak Wet Weather Flow.

Richmond-Sunset Water Pollution Control Plant

The Richmond-Sunset plant is the only one of the three City plants which treats both sewage and solids from only its tributary area and alternative processes need be concerned with the single plant alone. Additionally, reasonable land area within Golden Gate Park appears to be available for expansion and selection of alternative processes was not restricted by this limitation. The following eight alternatives were selected for economic analysis.

Alternative	Description
R1	Existing plant improvements with effluent pumping and an outfall extension.
R2	Existing plant improvements, effluent pumping and outfall extension plus dissolved air flotation treatment with a 33-50 percent recycle rate, a maximum surface loading rate of 2 gpm/sq ft not including recycle flow, and an air pressure of 60 psi.
R3	Existing plant improvements, effluent pumping and outfall extension plus chemical treatment with low dose ferric chloride (15-45 mg/l) polymer (0.25 mg/l) for 12 hours per day and salt water (1200-1500 mg/l NaCl). Ferric chloride polymer, salt water addition and sewage flocculation would be halted during periods of peak wet weather flow.
R4A	Existing plant improvements, effluent pumping and outfall extension plus biological treatment utilizing the activated sludge process with secondary sedimentation.
R4B	Existing plant improvements, effluent pumping and outfall extension plus chemical treatment with high dose slaked lime (240-280 mg/l), recarbonation and solids incineration with recalcining.

R5A	Improvements under R4B plus filtration, break-point chlorination and carbon adsorption.
R5B	Improvements under R4B with ammonia stripping, biological oxidation, filtration and carbon adsorption.
R5C	Improvements under R4A with nitrification, dentrification with chemical addition, filtration and carbon adsorption.

All alternatives were studied on the basis of ADWF of 28 MGD, PWWF of 70 MGD and total suspended solids at ADWF of 190 mg/l.

Southeast Water Pollution Control Plant

The high solids concentration in the influent and the requirement of treating the overflow from the thickeners for North Point solids must be taken into consideration in developing reasonable alternative treatment schemes. Of the many alternatives discussed and studied, the following eight seemed the most logical for economic analysis.

Alternative	Description
S1	Existing plant revisions with effluent pumping modifications and existing outfall improvements.
S2A	Existing plant, effluent pumping and outfall

improvements plus dissolved air flotation treatment with a 33-50 percent recycle rate, a maximum surface loading rate of 2 gpm/sq ft. not including recycle flow, and an air pressure of 60 psi.

S2B Existing plant, effluent pumping and outfall improvements plus chemical treatment with low dose ferric chloride (15-45 mg/l), polymer (0.25 mg/l) for 12 hours per day and salt water (1200-1500 mg/l NaCl). Ferric chloride, polymer, salt water addition and sewage flocculation will be halted during periods of PWWF.

S3 Existing plant, effluent pumping and outfall improvements plus biological treatment utilizing the activated sludge process with secondary sedimentation.

S4A Existing plant, effluent pumping and outfall improvements plus chemical treatment with high dose slaked lime (450-500 mg/l), recarbonation and solids incineration.

S4B Existing plant, effluent pumping and outfall improvements plus chemical treatment with high dose ferric chloride (100-150 mg/l), polymer (0.50 mg/l), and salt water (1200-1500 mg/l NaCl) with filtration.

S5A Improvements under S4A plus filtration, break-point chlorination and carbon adsorption.

S5B Improvements under S3 with nitrification, dentrification with chemical addition, filtration and carbon adsorption.

All alternatives were studied on the basis of ADWF 36 MGD, PWWF 70 MGD and total suspended solids at ADWF of 420 mg/l.

There are significant costs entailed in upgrading the plants to the highest levels. Total costs for all three plants including presently planned improvements amounts to from about $5 million to $183 million depending upon the alternatives selected. To this must be added the cost of providing additional wet weather capacity.

ESTIMATING THE FACILITIES

General

The major facilities in the proposed plans can be classified into five categories. They are:

(a) Retention basins

(b) Pipe conduits

(c) Treatment

(d) Tunnels

(e) Pump Stations

Each of these components has been estimated with consideration to location in the City and conditions such as geology, topography and water table.

In order that plans be evaluated on a comparative and realistic basis, all estimates were made on the base of 1974 dollars.

Summary

It is possible to draw some general conclusions regarding wet weather control facilities in the City which will circumscribe any design effort.

1. There is an optimum treatment capacity, storage volume relationship which is dependent upon the relative costs of each. For this analysis the 0.10 inch per hour rate appears to

be the breakpoint for optimum treatment for the range of storage volumes under consideration. Thus, all facilities are to be designed to accommodate this rate of withdrawal and treatment.

2. Storage volumes and treatment capacity must be distributed proportionally to the area served. The areal relationships then determine for various locations the storage volume sizes required at each site. This factor coupled with the economics of construction for various structures and minimum economical sizes constrains the number and location of upstream storage facilities.

3. Cost estimates for various facilities at various locations have indicated that the following facility locational disciplines must be followed to maintain a least cost system:

(a) Retention basins represent the least-cost storage option in shoreline areas and in sand;

(b) Tunnels provide the least expensive storage when constructed in sandstone, chert, basalt and serpentized peridotite.

4. Treatment volume may be counted as storage if the facilities are filled and evacuated as rate demands require. The maximum available storage volume achieved in this manner varies from 10% to 2% of the total required storage for the overflow frequencies studied. For the purposes of this report any incremental benefit of this volume was assumed to be minimal and

within the overall accuracy of the system sizing constraints. Thus, treatment volumes were not considered when accounting for total required storage.

5. Design rates of flow for treatment facilities used in this report are as follows:

Plant Zone	ADWF (a)	PDWF (a)	AWWF (b)	PWWF (c)
NPWPCP	74 mgd	134 mgd	259 mgd	370 mgd
SEWPCP	24 mgd	57 mgd	177 mgd	305 mgd
RSWPCP	27 mgd	54 mgd	188 mgd	325 mgd
Total City	125 mgd	245 mgd	624 mgd	1000 mgd

(a) values projected for the year 2020.

(b) based upon median hourly WW rate plus ADWF.

(c) based upon peak combined rate of 0.10 inches per hour equivalent rainfall.

6. To develop optimum benefit to the relief of existing inadequate sewers, upstream storage facilities should be located at points intercepting the upper one-third of the drainage district.

7. To derive maximum utilization of facilities for wet weather control, operation must be based upon an unattended intelligence system programmed from historical data collated to identify rainfall - storm variables affecting the decision making process as follows:

(a) initial location, area of influence, and time of storm inception.

(b) direction and speed of frontal movements if any.

(c) development and regression of rainfall rates and area relationships to apportion rainfall masses.

(d) mass releases and storage of rainfall derived runoff masses.

(e) response of the total system to each of the above parameters.

Oceanographic Discharge Parameters

Studies of the conditions of the bay and ocean with regard to the ultimate disposal of both the treated dry weather and wet weather wastes from San Francisco have been conducted over a full year of oceanographic conditions. Measurements have included both physical and ecological parameters for the purpose of developing design criteria for effluent disposal. As a result of this effort, criteria for discharge have been developed which reflect existing oceanographic conditions and which can be transposed to reflect future conditions.

The factors governing the design, location and successful performance of submarine outfall discharges may be divided into three classifications: oceanographic, ecological and physical. Oceanographic design criteria include those physical oceanographic factors such as currents and water density which influence the performance of an outfall. Ecological design criteria define the conditions which the discharge must meet to avoid a harmful effect on the marine environment. Physical design criteria refer to factors such as waste composition and flow rate, and the characteristics of pipelines and diffuser systems. In essence the physical criteria are those which may be manipulated while the oceanographic and ecological criteria are design constants.

Oceanographic Design Criteria

The objective of submarine outfall discharge is the disposal of treated wastes in a manner which will meet the water quality objectives and requirements of the regulatory agencies and in so doing have a minimum impact on the receiving waters. The potential locations and subsequent designs of outfalls from San Francisco meeting this objective are strongly influenced by the oceanographic characteristics of the Central Bay and the Gulf of the Farallones. Oceanographic design criteria which pertain to discharges to both the Central Bay and to the Gulf of the Farallones are as follows:

1. For dry weather discharges, the fall season represents the design condition because:

    a. Water clarity is greatest.

    b. Surface net advection is lowest.

c. Density stratification is least pronounced because of low fresh water inflow, and the tendency of an effluent field to rise to the surface is greatest.

d. Atmospheric and water temperatures are at the annual high, and recreational use of the shore areas is likely to be the greatest.

2. For wet weather discharges the winter season represents the design condition for the obvious reasons. During the winter period of high fresh water runoff:

a. Water clarity is lowest

b. Surface net advection is highest

c. Density stratification is most pronounced

Oceanographic criteria which apply only to the Gulf of the Farrallones may be summarized as follows:

1. To achieve a continuously submerged effluent field an outfall diffuser must be located outside the bar in 80 ft or more of water.

2. A surface field released at any point inside the bar in a water depth greater than about 60 ft will be advected seaward.

3. The bar area itself is too shallow to permit either surface installation of a major pipeline or good initial dilution for a major effluent discharge.

4. Effluent discharged through a properly designed diffuser located west of the mouth of the Golden Gate will have no measurable effect on the bay.

5. Floatable material released west of the mouth of the Golden Gate will not enter the bay.

6. Any dry weather discharge to the Gulf of the Farallones should be located at least one mile offshore to;

a. avoid the near-shore currents which have a net bayward displacement,

b. place a surfacing field beyond the limit of easy visibility from shore, and

c. increase the minimum shoreward travel time.

7. A wet weather discharge might suitably be made less than one mile offshore near the mouth of the Golden Gate in an area where the effluent field would be entrained in the westward-moving surface water mass.

8. An outfall and diffuser in the high current and unstable bottom area near the mouth of the Golden Gate will cost more per unit of length than in areas of lower currents.

9. The dissolved oxygen resources of the surface waters of the Gulf of the Farallones are greater outside the bar than inside the bar.

10. A surface effluent field in the Gulf of the Farallones will disperse horizontally in accordance with the formula

$K = 20L^{4/3}$ sq. ft/hr., and vertically

$K = 10$ sq. ft/hr.

Oceanographic criteria which apply only to the Central Bay may be summarized as follows:

1. Net advection of the surface layer in the Central Bay is seaward at all times of the year. Seaward advection is weakest in the summer and fall and strongest during periods of high runoff.

2. Surface advection in the bay south of the Golden Gate Bridge is much weaker than in the Central Bay, but still has a net seaward vector at most times and stations.

3. Surface drift of floatables released in the mid-Central Bay is seaward at all seasons. No significant deposition will occur along the bay shoreline, and the distribution along the ocean shoreline will be approximately the same as for an ocean release.

4. Density stratification is sufficient to keep an effluent field submerged most of the time at initial dilutions of 100 to 1 or greater. At times in summer and fall, however, there is no density gradient, and the effluent field will surface.

5. A surface effluent field in the Central Bay will disperse horizontally in accordance with the formula $K = 33L^{4/3}$ sq. ft/hr, and vertically at $K = 20$ sq. ft/hr.

6. Dissolved oxygen resources of the Central Bay are in excess of the lower limiting values established by the Regional Water Quality Control Board and recommended by Bay-Delta Program.

7. Tidal exchange at the Golden Gate brings 20 to 30 X 10^9 cu. ft. of new ocean water into the Central Bay each 25-hr tidal cycle during the dry weather months, and up to twice that amount in wet weather.

8. Tidal exchange at Alcatraz Channel brings 15 to 25 X 10^9 cu. ft. of new water past that site each 25-hr tidal cycle in dry weather months.

Ecological Design Criteria.

A number of water quality objectives have been set forth by the control agencies to prevent adverse ecological effects from marine waste discharges. Because of a lack of definitive information on the acute and chronic effects of waste discharges on marine biota, these water quality objectives are generally couched in general terms which require subjective evaluation. The results of the field and laboratory investigations over the last two years, together with the results of similar investigations by others, have been used to develop design criteria for use in planning for waste discharges in a manner which will avoid adverse ecological effects consistent with the water quality objectives of the regulatory agencies.

The ecological design criteria can be separated into two parts. First and most important are those criteria which relate to the dilution necessary to avoid adversely affecting the marine biota.

If these criteria are met, the second part, related to preferred location of discharge, is less important. The first four criteria listed below relate primarily to dilution, and the last four to location or method of construction. All criteria are based on the assumption that future discharges will be equivalent in toxicity to dry weather chlorinated primary effluent. In translating the results of field and laboratory tests into the design criteria listed below, a factor of safety of 10 has been employed.

1. Where possible, effluent dilutions along the shoreline or in shallow water should not be less than 1000 to 1 for more than 24 hours at a time.

2. Gravid Dungeness crabs appear to be vulnerable to the effects of exposure to sewage effluent through reduced egg-mass viability. The benthos in areas where gravid crabs are present should not receive sustained exposure to effluent in dilutions less than 500 to 1.

3. Plankton and fish populations should not be exposed to effluent dilutions less than 100 to 1 for more than 24 hours or less than 200 to 1 for long term exposure.

4. Deposition of sewage solids on the ocean floor should be avoided. Settled material of sewage origin has been demonstrated to have a negative effect on benthic populations.

5. From the standpoint of protecting the marine ecosystem in the Gulf of the Farallones, a surface effluent field is preferable to a submerged field for two reasons:

a. A surface field will be transported away from intertidal areas.

b. A surface field provides the greatest factor of safety for protection of the benthos.

This is particularly true during the winter season when gravid crabs are migrating shoreward.

6. As a general rule, dry weather discharges should be limited to about 100 mgd per outfall. Evidence from Southern California outfalls shows that larger discharges have had a measurable adverse effect on the marine environment.

7. Since rocky intertidal areas have a greater diversity and productivity than sandy beaches, a preferred location for an outfall in the Gulf of the Farallones would lie south of a line extended westward along the centerline of the Golden Gate. The sandy beaches to the south are not productive clamming areas and are not likely to be protected for the taking of shellfish.

8. Submarine pipelines and diffusers in the Gulf of the Farallones should be constructed in a manner which will not impede the periodic shoreward migration of breeding Dungeness crabs and certain other benthos.

These criteria can then be translated for each area of potential discharge around the periphery of the City.

North Point Zone Wet Weather Outfalls

Treated wet weather design flow (at 0.10 inches per hour treatment rate) from the North Point zone is about 370 mgd. Of that amount 200 mgd can be discharged through the planned dry weather outfall for the NPWPCP. There are only two basic options to be considered for the discharge of the remaining intermittent wet weather flows: either the intermittent wet weather flows will be discharged through the dry weather outfall, or they will be discharged through a separate outfall.

As presented in the oceanographic design criteria, it is a basic premise that environmental protection is best achieved by discharging wet weather flows in a manner which will result in a surface effluent field. Seaward advection of the surface layer will then move the field out of the bay in the shortest possible time. Design in accordance with this premise, however, will require the approval of the regulatory agencies. Ultimately, the plan which must be followed is the one which meets the requirements of the Regional Water Quality Control Board. A surface field will of necessity have a low initial dilution and will not meet the Regional Board's present objectives for turbidity and color at all times.

Discharge of intermittent wet weather flows through the dry weather outfall will of necessity result in high initial dilution. If a separate outfall is constructed for wet weather flows, it may be designed either to produce a surface field or to achieve maximum initial dilution.

Combined Outfall.

Only a cursory analysis is required to show that use of a single outfall to carry by gravity both the wet weather flow of 370 mgd and the present dry weather flow will create several unsatisfactory conditions.

The outfall pipe would be larger than 10 ft. in diameter, and the velocity at present peak dry weather flow would be less than 2 fps. The avoid an excessively long diffuser section the diffuser ports would have to be about 5-inch diameter, which would give initial dilutions of less than 100 to 1 at peak flow. At flows less than average dry weather, the head loss across the large ports would be so small that uneven distribution would occur and results of initial dilution would be unpredictable. If a single outfall is to be used for both dry and wet weather flows from the North Point area, therefore, it should be planned on the basis of effluent pumping at the North Point plant.

The outfall and diffuser designed for the North Point Plant can satisfactorily dispose of both the present dry weather flows and the plant hydraulic capacity of 200 mgd without effluent pumping. The same outfall, with a pumping head of 20 ft, can also satisfactorily convey and disperse the peak wet weather flow of 370 mgd. The diffuser would provide minimum initial dilutions in excess of 100 to 1 for the peak wet weather flow of 370 mgd.

Separate Outfalls.

If a separate outfall is constructed for the discharge of intermittent wet weather flows, it will have a capacity of 170 mgd. From a water quality standpoint the location of the second outfall is not of major importance. Dispersion characteristics and water quality are not substantially different for the entire waterfront area of the North Point zone. However, seaward displacement will be more rapid from stations closest to the Golden Gate, and the outfall should be located as far westerly as economics will permit. For greatest protection from damage by ships' anchors, the outfall should be located either west of Pier 37 or within the cable area which extends from Pier 9 to Pier 24. The westerly location again is preferable.

If the wet weather outfall is to be designed for maximum seaward displacement, the depth of water over the diffuser need only be sufficient for navigational purposes. For the dry weather outfall, this depth was set by the San Francisco Port Commission at 55 ft. The outfall must extend from shore far enough to entrain the effluent field in the main body of the westward-moving surface water mass. At most locations a length of 2500 ft should be sufficient. Depending on the head available, the outfall pipe diameter would probably be between 72 and 96 inches.

In order to insure that the effluent will rise to the surface, the diffuser would be designed to produce an initial dilution of about 10 to 1. With greater initial dilution the effluent would reach stability with the stratified winter water mass at some point below the surface, and seaward advection would be impeded. The diffuser would have a small number (5 to 10) of large diameter ports spaced widely and discharging straight upward.

If the wet weather outfall is to be designed for maximum initial dilution, the same design parameters used for the dry weather outfall will be employed. In this case preference will be given to depth of water over the diffuser. Water depths of 70 ft or greater are generally available within 3000 ft of the shoreline all along the waterfront from Rincon Point to the Marina.

Treatment Required for Wet Weather Flows. Intermittent wet weather flows will require treatment for substantially complete removal of gross settleable and floatable material, followed by disinfection. Whether or not additional treatment is required will depend on decisions which must be made by the Regional Water Quality Control Board regarding effluent parameters, and turbidity, discoloration, and maximum initial dilution as opposed to maximum seaward advection. With maximum initial dilution and a submerged field most of the time chlorination will be required to achieve about a 99 percent reduction in coliforms. For maximum seaward advection with a continual surface field a 99.9 percent coliform reduction will be necessary.

SOUTHEAST ZONE

The Southeast water pollution control plant has a submarine outfall and diffuser system with a capacity of 70 mgd, the existing hydraulic capacity of the treatment plant. A steady-state dye study of the outfall, showed that the diffuser achieves an initial dilution of 100 to 1 or better most of the time. Dilutions less than 1000 to 1 were not observed farther than 1000 ft from the diffuser. The minimum dilution observed in an effluent B.O.D. was about 50 to one. Because the Southeast zone has a functioning dry weather outfall, less attention was paid to the collection and analysis of oceanographic and ecological data for the bay south of the San Francisco Bay Bridge. As a result, recommendations for discharges in that area cannot be as specific as those for the Central Bay and the Gulf of the Farallones. Nevertheless, some general conclusions can be drawn from the information available.

South of the Bay Bridge the effect of tidal flushing is less than in the Central Bay. The farther south one goes the less effective tidal flushing becomes. On the other hand, the shallows and shoreline areas south of Alameda and Hunters Point fall in the category defined by the ecological studies as requiring the highest dilutions for protection of the environment. It is not likely that the ecological design criteria can be met during prolonged periods of high wet weather flow from the Southeast zone. The criterion for 1000 to 1 dilution in shallows and shoreline areas was established for an effluent with a toxicity equivalent to the present dry weather discharge from the City of San Francisco's three primary treatment plants, and includes a 10 to 1 safety factor above the limit of detectable adverse change in the most sensitive organisms. The dilution criterion provides a useful yardstick for measuring the relative merits of different discharge locations and methods. In the case of the Southeast zone the available discharge sites must be classed as less desirable than those for the North Point and Richmond-Sunset zones. This does not mean that the discharge of 300 mgd of wet weather flow from the Southeast zone will cause any serious, or even any detectable, ecological damage to the areas of concern. It does mean that if wet weather flow is determined to have a toxicity equivalent to dry weather flow the safety factor protecting against adverse change will be reduced.

Net seaward advection of the surface layer in the bay waters adjacent to the Southeast zone is lower than in the Central Bay, but still has a component of several miles per tidal cycle at most locations. This net seaward advection is sufficient to make a significant reduction in residence time within the bay for those wastes discharged on the surface. Since there is an obvious advantage of maintaining the shortest possible residence time in the bay during the period when waste discharge to the bay is at a maximum, it is recommended that intermittent wet weather discharges to the bay from the Southeast zone be made through outfalls designed to produce a surface field.

As in the case of North Point zone, the basic treatment required for intermittent wet weather discharges from the Southeast zone will be substantially complete removal of settleable and floatable material plus chlorination. For maximum initial dilution a 99 percent reduction in coliforms will be necessary, and for maximum advection (surface field) a 99.9 percent reduction will be necessary. Additional treatment may be required by the Regional Water Quality Control Board as noted for the North Point zone. Any additional treatment which reduces the toxicity of the effluent will reduce correspondingly the necessary dilution of effluent in bay water.

Richmond-Sunset Zone (Gulf of Farallones)

The disposal of wet weather flows for the Richmond-Sunset Zone involves several special considerations. Basically, these all relate to the range and magnitude of wet weather flows being considered. In order to achieve good performance at minimum flow and reasonable head loss at high flow a submarine outfall and diffuser is limited to a flow range of about 10 to 1. With special design provisions this range may be broadened slightly, but most of the wet weather options for the Richmond-Sunset zone are for flows too large to be included in the same outfall with the dry weather flow for that zone alone. Each wet weather option is therefore considered independently, and possible combinations will be considered in Chapter VI.

The oceanographic data for the Gulf of the Farallones show a massive seaward component to surface advection during the wet weather period. The phenomenon is strongest when the delta outflow is the highest and the receiving water is most strongly stratified, but persists around the mouth of the Golden Gate all year. As explained in discussions of wet weather flow from the North Point and Southeast zones, it is a basic premise of this report that large wet weather flows can best be dispersed by discharging them into the seaward-moving surface layer. This will serve the following purposes:

1. Large wet weather flows will be rapidly advected seaward and away from the ecologically sensitive shoreline areas. During the height of the rainy season surface flow through the Golden Gate is continuously seaward.

2. The large wet weather flows will be entrained in the low-density surface layer and will be slow to diffuse vertically. This factor will give additional protection to the benthos.

3. Wet weather flow will be dispersed in a different regime and will not compete with the dry weather discharges for dilution capacity. The dry weather discharges, which must receive high initial dilution, will not rise to the surface during the stratified flow conditions which normally accompany large wet weather discharges.

It is true that, inevitably, the runoff from San Francisco's first storm or two of the season will occur during dry weather conditions in the Gulf of the Farallones. The first storm or two will therefore create a highly visible effluent field which will be displaced seaward, though not as rapidly as runoff during the height of the rainy season. However, the stratified flow regime will build up within a few days after the start of heavy and prolonged rainfall, which produces the discharge conditions of principal concern, and with the increased flow stratification comes higher background turbidities in the receiving water and increased net seaward advection.

To achieve the objective of a surface field with wet weather discharges, outfalls must be deliberately designed for low initial dilution and high subsequent dilution. Inside the bar, for example, a wet weather discharge will not rise to the surface if it receives initial dilution greater than about 10 to 1. This indicates that diffusers for large wet weather flows should have a small number of large diameter, widely spaced diffuser ports. As noted earlier there are two areas for which discharge may be considered one zone inside the bar and one zone outside the bar. These are shown in Plate IV-39. No discharges should be made in the eddy areas off Baker's Beach or outside of the areas shown in Plate IV-39.

Treatment Required for Wet Weather Flow

All intermittent wet weather flows will require treatment for substantially complete removal of gross settleable and floatable material. For disposal in the manner described above, no further treatment is required to protect the marine environment. Wet weather discharges to the surface inside the bar will probably require chlorination to achieve a 99.9 percent reduction in coliforms. Wet weather discharges with high initial dilution inside the bar, and wet weather discharge the surface in the area south of the bar, will require chlorination to achieve about a 99 percent reduction of coliforms. The Regional Board's attitude toward effluent parameters and turbidity, discoloration, and maximum initial dilution as opposed to maximum seaward advection will determine whether additional treatment is required.

CHAPTER VI

ALTERNATE SOLUTIONS

Introduction

The previous chapter has discussed the various aspects of the analytical approach used in arriving at design criteria for wet weather control facilities. The control facilities developed are a combination of a total treatment plant capacity of 1000 MGD, and a storage volume which varies with the degree of control desired. The set of combinations is designated as the Master Plan and is the recommended control system.

The Master Plan presented considers all factors involved in a scheme for control and was selected because it represents the best conceptual design in terms of control range and flexibility consistent with least costs based upon the available data developed within the scope of this report. Further refinement will occur with the completion of the recommended continuing studies and the resultant broadening of the data base. The Master Plan is developed in three general stages which can be expanded in terms of total storage volume to develop overflow frequencies from 8 times per year to once in five years. This scheme was based on the data developed relating runoff to storage and treatment and presented in Chapter V. Control of 90% of the combined overflow is effectuated by the use of a combined storage and treatment volume of less than 2% of the total combined flows.

The following conclusion regarding treatment plant capacity was reached in Chapter V:

> There is an optimum treatment capacity, storage volume relationship which is dependent upon the relative costs of each. For this analysis the 0.10 inch per hour rate appears to be the breakpoint for optimum treatment for the range of storage volumes under consideration. Thus, all facilities are to be designed to accomodate this rate of withdrawal and treatment. The equivalent plant capacity for the 0.10 inch per hour rate is 1000 MGD which is the ultimate Master Plan treatment rate.

Four storage volume alternatives were developed, in Chapter V, to control overflows with increasing storage volume resulting in a diminishing average number of overflows to be statistically expected in any year. The alternatives developed provide storage to control overflows to 8 overflows per year, 4 overflows per year, 1 overflow per year, and 1 overflow each 5 years, (Alternatives A through D respectively). Each alternative includes the arrangement and location of facilities within the constraint of the existing sewerage system such that the least overall cost is achieved considering collection, transport, treatment and discharge of flow.

Scope

Five basic items which determined the final selection of the Master Plan were:

(1) Control

(2) Required Treatment

(3) Operational Feasibility

(4) Acceptable Discharge Location

(5) Cost

With the possible exception of operational feasibility, cost is the overriding factor as the other items are reflected in the cost of any solution.

A description of the recommended Master Plan set follows, with capital costs for both the wet weather and dry weather portions of the system. No specific alternative of overflow frequency is recommended as the selection is a matter of policy entailing significant expenditures of public funds within the range of the alternatives presented. The alternatives presented do represent the recommended range of effective control levels attainable.

Storage Volume Concepts

The recommended planning set for the control of wet weather overflows was conceived within the context of the basic regimen previously described. It is a plan that includes consolidation of 43 existing overflow outfalls to 15 via shoreline retention of flows by both basins and tunnels depending on the location, and a new water pollution control plant. The conceptual plan set forth is shown in Plates VI-1 through VI-4. It was conceived in such a fashion that by increasing the storage capacity of the various components or adding additional components the degree of overflow protection can be increased from 8 overflows per year to 1 in 5 years and the annual untreated overflow can be decreased from the present 780 million cubic feet to 3 million cubic feet. This would represent a control of 99.5% of the total runoff. Alternate A of the envisioned plan, which is modular in concept, provides a protection of 8 overflows per year. Alternate B controls all but 4 overflows per year. Alternates C and D provide additional storage to reduce the overflows respectively to 1 per year and 1 per 5 years.

Each alternative contains three major phases of construction of facilities, phase 1 which provides the facilities for the western and northern beaches, phase 2 which picks up the northeastern water front areas and phase 3 which provides facilities for the remainder of the eastern shoreline. Expansion from one alternative to the next can be accomplished by adding storage modules to the previous alternative.

At the inception of the study for the location of storage basins, investigation was made on the basis of placing all the storage volume at the shoreline at points of outfall consolidation in order to contain flow from the total drainage area made tributary thereto. Two general methods of storage were examined. They were by retention basins or storage tunnels. A detailed analysis of the cost of tunnels in various materials and locations in the City was conducted and many different types of retention basins were analyzed, including reinforced concrete basins. The unit prices developed for these storage facilities were used for the costs presented in Chapter V. When the data is plotted on a cost per unit of volume of storage, Plate VI-5, it is demonstrated that tunnels at the shoreline, or in areas where water is present, are more costly than retention basins for any volume analyzed. Thus retention basins are more economical than tunnels for shoreline storage. Similarly upstream basins cost less per unit of volume than shoreline basins. (Curve 1 vs. Curve 2 on Plate VI-5). Based upon this it was determined to minimize shoreline storage. Another reason leading to this decision was the fact that storage at the shoreline requires pumping to transport flow to a plant for treatment. This would be true around the periphery of the City. Based upon this concept the solution incorporates a maximum of upstream storage for the control of flow in conjunction with peripheral-basins to intercept and contain flow from areas too low to be stored at higher elevations.

The several factors related to location of upstream storage include the location of major transport sewers. It was also beneficial to situate basins at plateau locations such that they could be drained by gravity into the downstream existing system.

Again referring to cost, curves 1 and A through F, Plate VI-5, the unit price for tunnels in sand are greater than that for retention basins. Thus, no benefit would result in utilizing storage tunnels on the west side of the City as most of the area is sandy. In areas on the West side of the City where there is material other than sand, the individual required storage volumes are such that retention basins are less costly than tunnels. However in the case of the upstream areas on the easterly side of the City the option for tunnels in cases of storage volume in excess of 0.6 million cubic feet are economically beneficial. Regardless of type of storage, either the tunnel or the retention basins must be capable of being drained to the sewerage system. The location of a site for a retention facility was selected in so far as possible to be upstream of an inadequate portion of the transport sewerage system. The flow attenuation thus generated by the basin would serve two purposes; the first being the reduction of combined sewer overflows and the second being to reduce the flow rate in downstream sewers thus relieving their inadequacy. A further benefit can be derived relative to surface transport. The location of areas where surface runoff will commonly pool on the street surface during a high intensity storm are shown on Plate V-8. Upstream basins were located to relieve this problem where possible. The benefits of using upstream tunnels to store flow includes the option of using a tunnel to transport flow from one area to another. The fact that the tunnel intake is to be in an upstream area allows cross town transport of flow by gravity. The necessary head is developed by the elevation of the tunnel at the upstream end. This is a very important feature in the evaluation of the existing treatment facilities versus the cost of construction of a new treatment facilities for both dry weather and proposed wet weather treatment. The created option of inter-zone flow transport also allows consideration of many alternate sites for the establishment of a new plant or plants for the treatment of both dry weather and wet weather flow.

The desirability of using tunnels for storage of high level flow and the locations selected enabled a master cross town transport tunnel to be considered. Included with this transport tunnel which is of a minimum diameter to carry a 0.1 inch per hour rainfall on the tributary area, are the necessary storage tunnels. Storage is provided in large diameter tunnels up to 34 feet in diameter; the tunnel bottom contains a separate transport section. The tunnel cost study indicated that on the basis of the desired 5,000 foot minimum length the lower unit cost of the volume of tunnel was obtained when the diameter was equal to or greater than 32 feet. Flow will be dropped from the surface sewerage system in such a manner that sanitary flow, settleable and floatable materials will be directed insofar as possible into the transport tunnel. The storm flow at the selected locations can be committed to a storage tunnel. When desired a selected discharge rate from storage to the transport tunnel can be made. Included in the control mechanism will be the capability of isolating each or any combination of storage tunnels from the transport tunnel in order that one or more other storage tunnels may be emptied at a rate faster than 0.1 inches per hour for the tributary area. It then follows that when a portion of the City is receiving more rain than another, an appropriate control mode can be exercised. Schematic depictions of the proposed tunnels are shown on Plates VI-7 and VI-8. This mode and the operational controls are conceived to be automatic unattended via a master computer control system.

Two types of retention basins are also considered, shoreline basins and upstream basins. The concept involved at the shoreline basins is similar to that described for tunnels in that floatables and settleables will be taken, insofar as possible directly to an interceptor and thence to the treatment plant so as to minimize floatable and settleable materials in the basins. As depicted on Plate VI-9 a control structure will be constructed just upstream of the basin and contiguous thereto.

Incorporated in the design of all storage facilities will be an expansion chamber to slow the main sewer flow. A dropout will be located in this structure so as to conduct normal dry weather flow in the sewerage system to an interceptor. In the stilling chamber, the floatables will collect

in the scum chamber. The main flow will proceed through the chamber between a curtain wall and a weir type structure to a distributor channel from which it can be directed to any one or a number of compartmentized storage tanks.

Each individual tank will have its own distributor channel, which during low flow, will drop the influent to the bottom of the tank through a manifold of pipes. This manifold has another important use and that is the distribution of flow across the bottom of the tank for flushing. The bottom of each individual tank will be sloped toward a collection trough in the bottom which will lead flow to a control gate for exit from the tank. The flow will exit the tank and be pumped at a controlled rate to the interceptor system for treatment. At such time that the basin becomes full, provision is made for bypassing completely or for overflowing out the end of the tank to the existing sewer outfall. Included in the tank compartments will be a spray system to wash the interior surfaces of the tanks. A forced air ventilation system will be provided to provide air exchange which will exhaust into the interceptor sewer for release at the treatment plant. Control will be unattended automatic control via the master control system.

The upstream basins will operate in much the same manner. A control structure will be placed on the sewer ahead of the retention basin. Sanitary flow will be contained in the sewer and routed on to the sewerage system downstream. If a portion of the existing system must be removed, the sanitary flow, and floatables during wet weather, will be shunted to a bypass line to rejoin the existing system immediately downstream of the basin. During periods when the flow ration exceeds a predetermined quantity, 0.1 inches per hour, flow will enter the main distribution channel for the basin and thence be diverted by controls to any of the individual tanks. The internal features of the tanks are similar in concept to the shoreline tanks except that the outlet will be a gravity sewer, with a controlled flow rate, that will run on an interception grade to re-enter the existing sewer downstream. The individual tanks will each have a spray system for cleaning the interior surfaces and a ventilation system exhausting to the existing sewer system.

An example of the structure is shown conceptually on Plate VI-10.

All storage will be interconnected in a system which will allow a transfer of treatment capacity to service those areas with the greatest need during periods of non-uniform rainfall over the City.

This interconnection will minimize the probability of multiple overflow occurrences at different locations which cannot be prevented where zones are not interconnected. The interconnection of the City drainage and storage system will allow the alleviation of the cellular high intensity rainfall patterns which would otherwise result in multiple overflows at different locations and times. Such cellular patterns have been observed and are described in Chapter IV.

The interconnection will also allow some judgement to be exercised in allowing controlled overflows in those areas of higher dilution on lower priority receiving water usage. This can be accomplished again through the allocation of treatment capacity to areas of sufficient storage to contain overflows while allowing stressed areas which would overflow in any event to overflow under controlled conditions. This situation is the reverse of the above noted cellular pattern event.

A further advantage under this system which employs interconnection and the optimization potential resultant from unattended automatic storage and transport control and allocation is the accompanying potential for minimizing the total emissions during wet weather by maximizing the use of storage during the light to medium rainfall occurrences and attenuating the resultant higher flow rates to utilize the full treatment facilities to maximum capacity over an extended duration. This potential is apparent from an evaluation of the frequency of the use of fractions of the storage volumes provided under the four alternatives. As the size or volume of storage increases the fraction of infrequently used storage increases. Comparison of the percent of time that various volumes are used with total storage volume indicates that a fairly sharp decrease in frequently used volume occurs at about the storage required for four overflows per year.

Preliminary calculations and considerations indicate that any system for treatment that will afford significant pollutant removals must have a running time of at least 2 to 3 hours. This must be coupled with an initialization stage of about one hour.

Significant further verification and data base development must precede any final prediction of the resultant decrease in uncontrolled overflows. However, for conceptual design purposes the results of the analyses to date are used bearing in mind the inherent uncertainty of actual operational results.

In any event the uncertainty regarding uncontrolled overflows is increased with the number of separate control and treatment systems under consideration. Without central control and interconnection of the drainage and storage systems, the same facilities for storage and treatment that would limit overflows to 8 per year would experience from 24 to 128 overflows depending on the patterns of the storms of a given year. The automatic control system is therefore a vital element of the Master Plan.

Treatment Plant Concepts

Given the options of storage and transport described above and in Chapter V, the problem of determining what levels of treatment are required and where the plant or plants should be located and finally where discharge disposal may be accomplished remains.

The quality and character of the waters surrounding the City were described in Chapter V and more fully in reference

No. 24. Based on studies of the receiving water surrounding San Francisco, there were only two sites recommended for the combined discharge of the massive quantities of flow under consideration. One is at the northwestern corner of the City with an outfall extending to deep water in the channel near the entrance to the Bay. The main drawback to this location is that during periods of flood tide, a portion of the discharge might be transported back into the Bay proper. The other site recommended was a location in the ocean off the southwesterly corner of the City outside of the bar and which is completely described in Chapter V and reference No. 24. Given this context it was determined that for the combined total disposal plan, the location to be cited in this report is to the west and slightly south of the San Francisco bar. Selection of this area is based upon the following advantages: (1) the area is biologically relatively barren; (2) the depths selected are sufficient to provide the required dilutions for discharge with properly designed diffusers to meet the design criteria presented in Chapter V; (3) the option of provision for seasonal field variation between surface fields and submerged fields is possible through the use of dual outfall and diffuser facilities; (4) the shoreline is afforded maximum protection in terms of the dilution attained and the probability of effluent fields reaching shores; (5) if further protection is required as knowledge of the effects of disposal increases, then treatment levels may be increased without the necessity of overcoming existing background levels of pollutants as are existent in the bay.

Areas at the mouth of the Gate and near Blossom Rock have some, but not all, of the advantages noted. Areas south of the Bay Bridge are less desirable than any of the above locations. Insofar as dry weather discharge only is concerned, the two alternative ocean sites, inside the bar, and outside the bar, are equally acceptable. Thus, it is also feasible to consider two separate locations for discharge, wet weather south of the bar and dry weather north of the bar.

Consideration of these locations and of the required treatment facilities together with the gravity flow possibilities inherent in the storage system, leads to the alternative of consolidation of proposed wet weather and dry weather facilities. Presently the existing system accomodates some of the wet weather flow and discharges at three locations as described in Chapter IV. Possibilities considered for treatment extend from maintenance of the existing three treatment plants and providing one wet weather plant for intermittent use in each treatment plant zone to combining all treatment facilities into one plant at the southwestern corner of the City with subsequent discharge to the ocean. To avoid potential conflicts in judgements regarding levels of required treatment for various discharge locations, several treatment levels were considered with equivalent treatment evaluated for all flow at all locations.

The treatment process levels considered are described on Plate VI-11 .

The requirements for discharge from the existing plants do not distinguish between dry weather flow and wet weather flows and all existing discharge must meet the same requirements. Existing wet weather overflow requirements are very similar to the treatment plant requirements. Thus at this time similar levels of treatment are required for both dry weather and wet weather flows. Essentially this means that wet weather treatment for intermittent discharge must provide bacteriological protection of the receiving waters by effluent disinfection, aesthetic protection via the removal of grease and floatable materials and the reasonable control of discoloration and turbidity, and ecological protection through solids removals toxicant control by source containment and by proper diffusion of the effluent to provide a surface field. At times this field will exceed present requirements for turbidity and discoloration. However, the surface field provides the best ecological protection from the intermittent wet weather discharges consistent with the practical treatment levels to be attained. Dry weather continuous discharge will require better ecological and aesthetic protection which at this time can be accomplished by providing an outfall designed for maximum dilution. Given the treatment to accomplish these objectives and proper disposal with an offshore outfall, the existing and anticipated requirements of the Regional Board for wet weather discharges will be met.

Consideration of the required treatment for wet weather indicates that the treatment must be of the physical-chemical type to be feasible and that removals must be equivalent to or better than that which primary treatment affords. This is provided by the first level of treatment described on Plate VI-11.

Present dry weather requirements can be met with the same level of treatment together with a higher dilution outfall system. If higher levels of treatment are required for dry weather flows as a result of the adoption of more stringent requirements then the appropriate higher level will obtain as noted on Plate VI-11. However, it is not anticipated that higher than first level treatment for wet weather flows will be required. The treatment costs in this report are based upon the costs of upgrading the existing plants and providing first level wet weather treatment for flows exceeding their capacity or, in the instances where existing plants would be phased out, providing split level treatment in a new combination wet weather-dry weather facility. This will be described in detail later in this Chapter.

The first cost of constructing the matrix of treatment plant combinations is shown on Plate VI-12. The dry weather, wet weather cost fractions of the total cost are based upon the flow proportion of each accomodated by the plant combinations listed. These costs do not include any interim costs to provide for upgraded effluent where plants are to be phased out as indicated. Where multiple plants are indicated all are estimated based upon equivalent treatment levels, except where noted otherwise. As can be noted from the cost matrix the minimum total cost combination varies with the treatment level assumed. Selection of the least cost plan will depend upon the anticipated future requirements for each plant discharge location. In this regard, it is our understanding that in the near future federal requirements for funding assistance will require higher levels of treatment in terms of attaining a BOD removal of 85% or greater for discharges from dry weather facilities into the Bay. It is not anticipated that these requirements will apply to ocean discharges or to wet weather discharges from wet weather facilities.

If this requirement comes into being then the ultimate least cost solution is depicted by the scheme of abandoning the existing plants and constructing one new plant discharging to the ocean regardless of the level required for ocean discharge. Compliance with the requirement may require the expenditure of funds at the existing plants prior to the completion of any new plant as a result of the various priorities and limitations upon the availability of funds for implementation of the total Master Plan adopted. Plate VI-13 shows the costs relative to compliance with this requirement for the combinations of the three existing treatment plants and the costs which would not be recoverable following consolidation with wet weather facilities, as estimated on Plate VI-12. The additional cost of providing ocean outfalls has not been included in the first cost shown on Plate VI-13.

These costs would be included if compliance with the anticipated levels were required at the plants indicated prior to consolidation. Upon future consolidation the recoverable and non-recoverable costs are shown. If consolidation is adopted as the long-range goal with immediate major modifications to the existing plants to upgrade effluent quality then the non-recoverable costs shown must be added to the total consolidation costs as shown on Plate VI-12. For more stringent bay discharge requirements and first level ocean discharge treatment and adding the non-recoverable costs to the costs on Plate VI-12, the least cost scheme is represented by the single plant consolidation.

For other possible combinations the situation varies depending upon the level of expenditures required at the existing plants.

As cost differentials narrow annual operational costs become more important. Owing to the number of possible combinations the derivation of these costs was considered to be beyond the scope of this report.

For the purposes of this report the single plant total consolidation costs are used as they should be conservative and well within the accuracy of the total estimate to allow modification of planning as future State and Federal requirements become available.

The treatment level envisioned at this time and reflected in the costs presented for the Master Plan is first level treatment for both wet and dry weather flows with the potential for expansion to the split level treatment to provide higher levels for dry weather flows in the future if required. No higher level of treatment for wet weather flow is deemed necessary nor practical. If higher levels of dry weather treatment are provided in the future then utilization of the full system capacity in terms of treatment and storage attenuation could result in slightly higher combined removal levels than would be the case with separate facilities. The total cost for the wet weather portion of the four storage alternatives, with first level treatment, is summarized as follows:

Alternative A - $333 million

Alternative B - $396 million

Alternative C - $522 million

Alternative D - $665 million

The combined dry and wet weather total Master Plan costs are shown in Plate VI-14 which shows three schemes each with three levels of treatment for the four alternative flow rates. The 36 possible combined dry and wet weather treatment schemes range from $375 to $848 million.

Scheme 1, identified as most economical in the plate, would consolidate the dry and wet weather flow at one new treatment plant located at the Lake Merced site for disposal to the Ocean. This scheme would provide for only the absolute minimum of improvements necessary at the three existing plants until they are abandoned.

The second scheme, identified as RS and SE, would provide only for a wet weather facility at Lake Merced in the first stages and upgrade the three dry weather plants. At some relatively near future time the treated effluent from RS and SE would have to be diverted to the Lake Merced outfall for disposal. As wet weather facilities are completed raw sewage would be diverted to the new plant and present facilities abandoned. The unrecoverable cost associated with Scheme II (Plate VI-14) shown on Plate VI-13 is $31 million and $42 million for Level 1 and 2 treatment, respectively.

The third scheme would be identical to the second scheme with the addition of the North Point flow being diverted to Lake Merced for Ocean disposal. The unrecoverable cost associated with Scheme III (Plate VI-14) also shown on Plate VI-13 is $42 million and $53 million for Level 1 and 2 treatment, respectively.

The wet weather accomplishments under the four alternatives presented are shown in Plate VI-15. It is significant to note that improvements in pollution reduction beyond the A alternative are denoted only in the area of days of violation of receiving water bacteriological requirements. Other reductions listed vary only within the accuracy of the basis for their calculation and no significant difference is apparent. Using days of receiving water compliance as a measure of effectiveness the effect of diminishing returns is illustrated. Plate VI-16 shows unit costs per day of compliance attained based upon the wet weather costs for each alternative using total days of violation reduced from the existing condition. A marked increase is shown between alternatives C and D. Plate VI-17 shows the incremental day cost between alternatives. The effect of diminishing returns is more noticeable when evaluated in this fashion. Alternative B appears to represent the reasonable upper limit for control as the cost per day of compliance attained begins to increase sharply beyond this level.

A consideration of the benefits of improving the levels of dry weather treatment in the combined system versus the effect of providing additional wet weather storage is shown on Plate VI-18. This Plate shows that an increase in the level of dry weather treatment results in a reduction of the total amount of pollutants released whereas increasing wet weather storage with the same level of treatment results in insignificant increases in removals. The selection of level and control frequency should consider this aspect and recognize the inherent limitations to any wet weather control system as to total control.

Not considered in the development of the costs and alternatives presented are the differences in annual operational costs that may be attained with a single plant system as opposed to a multiple plant system. One such saving would be in total pumping costs. Flows presently pumped during dry weather are illustrated on Plate VI-19 with a comparison to what would be pumped under a single plant scheme. Presently only 6% of the total City dry weather flow is discharged without being pumped. Some flows are pumped more than once. Under a single plant scheme the total flow pumped would be reduced by about two-thirds of present values. The benefits to operational costs and reliability attained under such a system are obvious. Further savings may be incurred as a result of potential flow attenuation and storage optimization possible with the single plant interconnected system which would not be practical under any multiple separate plant scheme. Detailed operational costs are beyond the scope of this report as discharge requirements are not yet firm and the costs will depend largely upon the treatment levels necessary. As previously noted the costs used in this report and the solution described represent the capital cost range from first level to third level chemical treatment with a single plant at a site on the western shoreline south of the Zoo and henceforth referred to as the Lake Merced Pollution Control Plant.

The initial treatment scheme proposed is a split flow system. Treatment under this approach assumes the eventual development of first to third level chemical treatment for all or portions of the dry weather flow and first level treatment for wet weather flows. During dry weather the flow process would include primary sedimentation with low level chemical addition which could be followed by high level chemical addition and secondary sedimentation of chemical sludge.

Chemical sludge could be regenerated at the site or transported from the site with the primary sludge solids. Third level adjuncts could be added to the plant in the future to provide a high grade effluent suitable for reclamation usage.

During wet weather split treatment options would be exercised allowing optimization of total removals up to providing single stage chemical treatment for all flow with a fraction receiving further treatment as described above. All primary solids would be transported to the Southeast sludge treatment plant and treated for final disposal in landfill or by any other acceptable means. Following split level treatment various flow fractions in the Lake Merced plant may be cycled through a reclamation option or blended to provide intermediate effluent qualities. Reclaimed waste water may be used for ground water recharge in the area. A conceptual flow diagram showing the proposed treatment program is shown on Plate VI-20.

The site selected for the new plant is shown on Plate VI-21 and would occupy land now under the jurisdiction of the Park and Recreation Department, Federal Government and a portion leased from the City to the State. Present planning for the area has been incorporated into the facility design.

The plant envisioned would be designed to provide maximum multiple usage of the plant area consistent with long-range recreational planning efforts. It is anticipated that through modern design and effort side-by-side multiple usage of treatment facility land area will be possible. The experience in this regard at the Baker St. Facility in the Marina serves as a positive example of what can be accomplished. At the present time the conceptual design for the proposed Lake Merced plant has incorporated planned zoo parking facilities and some other multi-uses. A perspective view of the proposed plant cross-section and the plan layout are shown on Plates VI-22 through VI-24.

One alternative for discharge of treated effluent is through a dual outfall with a wet weather diffuser terminating about 2 miles offshore designed to provide a surface field for maximum surface dispersion and a dry weather outfall continuing to a point between 3 to 4 miles offshore in 60 to 80 feet of water terminating in a diffuser designed for maximum initial dilution. Plate VI-25 shows the proposed outfall location for this alternative.

The length and depth required for the dry weather outfall section will depend upon the anticipated combined dry weather peak design flow rate and the increases in treatment level which would result in a reduction in the discharge dilution criteria previously presented.

Selection of the final location will require further evaluation of the flow and treatment factors and for costing purposes the longer of the two outfall distances was utilized for this report.

An alternative outfall system which has approximately the same cost for the single plant scheme used in this report is shown on Plate VI-26. The decision as to which alternative should be selected will depend upon the program scheduling, funding available and the discharge requirements adopted by the regulatory agencies.

Either alternative will provide for maximum protection of all beneficial water uses including minimum ecological impact with optimal flexibility for long-term reclamation developments.

A final benefit that may be attained through this single plant location is the flexibility to further consolidate some of the smaller discharges now existent on the western peninsula coastline with the proposed plant.

<u>System Response Time</u>

As previously noted the storage and treatment system are interrelated in determining the amount of time the control system has to respond to each individual storm. A light rainfall and of relatively short duration may be diverted to storage and released at a low rate to effect the best possible removal efficiencies. Conversely a storm with a high intensity rainfall and of long duration will require that the treatment plant respond as quickly as possible to the full operational mode. The storage time available in retention for each of the Alternatives A to D when the treatment plant is operating at full capacity is as follows:

Alternative	Storage Capacity (Millions of cubic feet)	Time to Drain Storage @ 1000 MGD rate (hrs.)
Existing	0	0.25-0.50
A	9	1.6
B	18	3.2
C	33	5.9
D	56	10

An additional storage volume of approximately 2 million cubic feet is also available at the treatment plant in bringing the plant up to full operational status. This storage system capacity plus the transportation time to deliver the flow to the plant represents the maximum time period available to the treatment plant for responding to wet weather conditions.

This maximum response time must be reduced for control system design because if the total storage is allocated at the beginning of the storm the whole system would lose its flexibility to store for future increase in rainfall intensity rate beyond the treatment rate.

Thus the response time and commitment of facilities becomes a dynamic operation which must respond individually to each storm to produce the most efficient removals. To optimize system response a computer program must be in command of the whole system to consider the following input variables:

1) Rainfall intensity
   a) Temporal variations
   b) Spatial variations

2) Rainfall direction

3) Storage volume available

4) Sewer transportation time

5) Storage volume available at the treatment plant

6) Treatment plant rates

7) Selection of controlled discharge location

Considering all the control variables available to a computer in the wet weather control system the more data input will produce the best possible results. A system of rain-gauges within the City is approaching the operational stage and an advanced raingauge system located in the Farallone Islands, San Mateo County and Marin County is being investigated. This aspect of the Master Plan is critical to the operational viability of the system and must not be neglected in implementing the system as presented.

Alternative Solutions

Investigation of alternative solutions has included the extreme possibilities for control facilities including separation, total treatment with no storage and all storage using existing treatment. Each scheme was developed to the degree necessary to determine the feasibility of continued investigation. At the stage of development where it was obvious that further work along a particular approach was not beneficial or would result in an uneconomical solution, the solution approach was dropped.

From this matrix of possible solutions the recommended solution previously described emerged as meeting all criteria at least cost with the most flexibility to adjust to new advances during its implementation and developments. In arriving at this solution the following approaches were investigated and developed sufficiently to provide the building blocks for the recommended scheme.

The most obvious initial solution to any problem involving combined sewer overflows entails sewer system separation. This has been the classical approach presented in the past.

Any separation scheme involves the division of the existing combined sewerage system into separate dry weather and storm runoff sewer systems. The estimated total cost of accomplishing this was too great to seriously consider pursuing this aspect. Further, the fact that storm runoff would be collected and conveyed in a separate system would not preclude the eventual necessity for treatment of storm runoff before it is delivered to the receiving waters.

Recent studies have indicated that pollution from separate storm runoff is significant and treatment of runoff probably will eventually be a requirement so that on a long range basis it may be more efficient and economical to manage a single combined sewerage system than two separate collection systems. The possibility of increasing treatment capacity was investigated as a second simple scheme for control.

The all-treatment scheme makes no provision for separate storage facilities to dampen high storm flows with a subsequent release of smaller controlled flows to treatment facilities. However, the modular design of the treatment plants could allow them to provide this storage function if operated on a fill and draw mode when not operating at peak flows. It is doubted that treatment facilities can be activated and operated under transient conditions of rapidly varying flow. This plan was also determined to be too costly. Further, the problems of operation and maintenance would be of extreme magnitude owing to the inherent complexity of such wet weather plants. Problems of start-up and shut-down, year-round maintenance and finally the resolution of the sludge handling problems rule this solution out as a feasible approach.

Maximum storage using existing treatment rates was also investigated and found uneconomical so further study involved increased treatment capacity. All storage facilities might be placed on the shoreline. However, the cost of this would be greater than that of upstream and downstream combinations owing to the groundwater and soil conditions encountered at the shoreline. Further, no relief would be afforded inadequate sewers and all flow would have to be pumped to treatment.

The preliminary investigations of the alternative solutions of separation, increasing only treatment plant capacity, and maximum storage resulted in their abandonment as viable alternatives. The recommended solution of storage and treatment thus was selected for the Master Plan.

CHAPTER VII

IMPLEMENTATION AND SYSTEM MODIFICATIONS

Given the Master Plan described in Chapter VI, a system of staged construction is required which will be compatible with present needs relative to the dry weather facilities and provide for optimum progress in completing the wet weather control program. Additionally, the program must include and consider "in-system" improvements to the existing sewer system.

Improvements to the sewage transport system fall into six general categories, each of which will be discussed hereinafter. They are:

1. The use of upstream storage basins, strategically placed, to increase the effective capacity of lower portions of the 150 miles of the transport system.

2. The revision when warranted of official street grades to provide a designed surface transport system with the elimination of existing areas of runoff ponding.

3. An internal inspection program to intelligently evaluate the replacement needs of the small pipes, when manual inspection is impossible, in the system.

4. A controlled program of isolating the sewers of the interconnected network to prevent reversal of flow, and provide a means to predict the flow path of any tributary area necessary to establish hydraulic grade lines which conform to street elevations.

5. A program to phase out the two existing "separate" sewer systems in the City.

6. A revision of design criteria to reflect the information gained from the new monitoring system.

The wet weather requirements dominate the contemplated improvements to the system. The storage of flow, the consolidation at specific storage locations and the required treatment all are integrally related to the transport system. The facilities envisioned in all cases, are dependent upon the delivery of flow to a particular point, whether by the transport system or overland flow during very heavy storms. As presently structured, the existing system is expected to carry the flow up to the 5-year storm. The excess flow from storms of greater intensity is designed to flow overland until it either reenters the system or is dispersed to the receiving waters. The strategic location of retention basins on the transport system will reduce peak flows to the capacity of sections which formerly were undersized for a 5-year storm.

The location of the retention basins can be evaluated by the following:

a. Decrease in the calculated frequency of flooding on the downstream system.

b. The opportunity to connect overland flow corridors to the transport system.

c. Topographical features which permit gravity draining of the basin by a relatively short reach of sewer.

There is a need to place basins so as to pick up the overland flow portion of the storm. The hydraulic design of the streets should provide a means to carry flows during the more intense magnitude storms. Plate VII-1, developed as part of a surface flow routing study indicates a method of describing the magnitude and direction of surface flow transport. If surface routing is to function to direct flow to a basin site, the official grades of streets must be set in such a manner that sinks are not created but a smooth surface collection system which will drain downhill continuously must be created. Low end cul-de-sacs must be provided with facilities to transport flow overland to the surface collection system.

Prolonged erosion in the steep reaches or accumulated attack by chemicals coupled with increasingly heavy street loads eventually leads to physical breakdown of the conduit.

There are approximately 250 miles of sewers in the City over 80 years of age and a continuous replacement program is a necessity. In order to determine which lines should be replaced, field investigation is a basic requirement. Larger sewers, over 4 feet in diameter or height are inspected internally by visual examination. The smaller sewers in the City, the 750 mile collection system, must be inspected remotely by means of photography or television. The current annual 2½ miles amount of inspection is inadequate and will require 100 years to complete the inspection of the current pipe which is now 80 years or more in age. Without the needed funds for photography or television inspection of the small collector sewers a new roadbed and tracks may be placed over deteriorated sewers requiring subsequent excavation and expenditure of funds for sewer replacement which otherwise could have been accomplished simultaneously with the street renovations.

There is one major area of the City that is serviced by an interconnected sewer network and seven outfalls to the China Basin channel. The area is bounded basically by the Bay, Sansome Street, Market St., Tenth St. and Divisadero St. To complicate the issue, most of this system is also in the subsidence area of the City. Many of the sewers in this area have settled to such an extent that the original limited slope has diminished or in some cases reversed. Others have sagged to the extent that decomposing materials are held in the system contributing to the septicity of sewage, generation of sewer gas and odors.

Interconnections at almost each street intersection and hydraulic grade differences during low flows leads to reversal of the flows in some of the sewers and causes undesirable deposition of materials. Such deposition can cause flooding under storm conditions. During wet weather low velocity flows intersect high velocity flows and cannot enter until sufficient head is developed which may result in flooding. In order to provide a definable system that is manageable hydraulically continuous flow paths at all times must be provided.

There are two areas in the City that are served by a separate sewer system. The larger of these is located generally between Market and Howard, Embarcadero and Second Streets. The second is on the westerly side of the City at the foot of Vicente Street.

It is presently required that any building that connects to the sewer system in these areas construct separate side sewers to transport sanitary wastes to the sanitary system and roof leaders and drains to the storm system. Over the years, storm drains have obviously been connected to the sanitary system as indicated by a large increase in flow during periods of heavy rain. Maintenance of the systems is difficult and costly.

The "separate" district at the end of Market St. was initiated circa 1910. In addition to being constructed between 12 and 20 feet deep, it is also in the subsidence area of the City. At times, lines have become completely closed with deposition. Because of the extreme depth of these lines, it is difficult to maintain them. At times because of this depth, the equipment required is beyond the means of maintenance forces, necessitating the use of contractual assistance. The City's design criteria provides a 5-year storm hydraulic grade line about 1 ft below gutter level at any point in the sewerage system. The concept of providing a sanitary flow grade line 8 to 20 feet below street level with a simultaneous high storm grade line in part of the City is not in the City's best interests. The separate systems are tributary to a pump station to lift the flow to a treatment plant. Because of excessive flow during rain these sewers overflow and discharge to the receiving waters.

In order to eliminate the additional expense of maintenance and obviate the current need to reconstruct portions of the sewers at these low elevations, the separate system should be phased out beginning with the extremities of the system and progressing toward the concentration point at pump stations. This will require an expenditure on the part of property owners to reconnect to the existing storm sewer. Where the sanitary sewer is lower than the storm sewer, plumbing modification within some of the buildings will be required. The cost of the side sewer construction, if any, should be borne by the City and the internal plumbing costs by the building owner. The City's costs will be offset by the reconstruction costs otherwise necessary.

In the Vicente zone a proposed wet weather facility will eliminate the need for any prompt action toward separation of that separate system. Abandonment in phases as sewers are replaced is recommended for this area.

In order to provide the City with a sewerage system encompassing the latest transport techniques, the present method and criteria of design should be changed in an orderly fashion. Of foremost importance is the method of calculating flows in any part of the sewer system. At present, the rational formula is used to approximate flows. As the data acquisition system described in Chapter VIII supplies more transitory flood wave data, the procedure of design should change to a hydrograph method rather than the conventional empirical "Rational Formula".

In general, all hydraulic losses, both energy and momentum, should be considered in the design of the sewerage system. The system design will be based on the complete system hydraulic profile, utilizing the energy grade line as the control value.

With regard to actual sewer installation, the more important criteria are as follows:

(a) the minimum size sewer shall be 12 inches in diameter.

(b) where practicable, the minimum cover shall be four (4) feet.

(c) direct drainage of basements below culvert elevation will not be allowed or only at the risk of the property owner.

(d) side sewers in buildings to be demolished shall be inspected by the City prior to reconnection to any replacement structure.

(e) sewers may be laid on horizontal or vertical curves, the two not coincident, in order to take hydraulic advantage of the natural topography.

(f) manholes shall be constructed at the ends of curvilinear portions of sewers or at intervals not more than 300 feet.

(g) in order to uniformly merge flows of different momentum and energy levels, design will provide for velocity maintenance throughout the merging zone.

Wet Weather Facilities - Staging

Concurrent with the internal system changes in design, policy, and on-going construction, the main wet weather control system construction must be implemented. The basic concept of staging is to develop facilities to protect areas of more intensive recreational usage first, followed by areas of lesser use. In this regard the staging in general gives first priority to the Northern water recreation areas and the Oceanside areas followed by the Eastern side of the City.

A stage breakdown is presented on Plate VII-2 which presents a more detailed staging plan and the dollar expenditure envisioned for each stage for the four Master Plan alternatives.

Each stage consists of a block of components which must be completed to function for the unit area served. Stage order may be varied as needs or priorities change. This is particularly true of the treatment plant stages as discharge requirements for dry weather flows are in a state of rapid flux and as requirements become more stringent the need for a single high level treatment plant becomes more pressing. Stage timing will be dependent upon the availability of local funds and grant assistance. Obviously if grants are available the time for completion can be compressed within the constraints of local funding. This aspect will be covered more thoroughly in Chapter IX.

Each of the sixteen stages is discussed briefly in the following paragraphs. The specific descriptions of typical facilities are in Chapter VI, together with discussion of the operational features of the structures. The sizes of the storage facilities will vary depending on the Alternate selected. Plate VII-3 presents the volume requirements of each of the storage facilities for the four Alternates. Plate VII-4 shows the proposed dimensions of the retention facilities to provide the required volumes under each Alternate, taking into account the physical constraints of available area and hydraulic requirements. Wherever possible, structures were designed to be modular, so that a building block approach could be used to expand the facilities if required.

Coupled with treatment needs is the need for industrial waste source control. This is particularly true for toxic constituents which are not amenable to conventional treatment processes. Heavy metals, pesticides and organic compounds fall into this category of potentially toxic compounds which must be given source control. Materials which cause higher operational costs must also be controlled in so far as possible. Plates VII-5 ,VII-6 and VII-7 show daily influent composites of heavy metals concentrations over one week of sampling at each treatment plant.

A comparison of a weekly effluent composite of selected heavy metals is tabulated in Plate VII-8 , which also contains a column showing RWQCB regulation for another discharger. The present effluent information is of interest but not directly applicable as treatment will be upgraded at the new Lake Merced plant to reduce these concentrations.

The concentration of heavy metals in the influent of the existing plants is of greater value as it is expected that the municipal or sanitary portion will remain basically unchanged while the industrial portion will decrease. The expected decrease is due to enactment of the City's Industrial Waste Ordinance. The influent concentration tabulating existing conditions at the plants, and those portions attributable to the following origins: drinking water, municipal or sanitary, and industrial. The industrial origin portion will be subject to the Industrial Waste Ordinance.

The City's Industrial Waste Ordinance is designed to increase the charges to industrial discharger of heavy metals, among others, to either give the dischargers an incentive to reclaim the metal or to provide the City funds to pay for the additional cost required to handle these toxic elements.

From the heavy metal concentrations, even including the variations between daily and weekly influents composites, it is apparent that industrial batch discharges are contributing significant quantities to the sewer system. As no fish kills have been observed in or around San Francisco it is doubtful if any acute toxic effects are being exerted through these batch discharges. However for long term control of chronic effects industrial waste control is necessary. Plate VII-9 tabulates the possible influent heavy metal masses which might be controlled through industrial wastes source control. From about 30% up to 96% of the influent heavy metals might be controlled in this manner.

STAGE 1. The Marina/North Waterfront/Financial Area. This area represents in a smaller scale most of the features characteristic of San Francisco. Starting with the Marina yacht harbor and moving eastward along the shoreline, the facilities range from recreational areas in the Marina and Aquatic Park, to commercial fishing in Fisherman's Wharf, through waterfront usage in the piers along the northern waterfront down to the Ferry Building. There are 10 outfalls existing in this area which overflow during wet weather. Southward from the shoreline on the western end, the land use changes to high density residential as it goes up into the hills overlooking the Bay. Eastward, the character changes into mixed commercial and residential use to , finally, the downtown section consisting of a concentration of business and financial institutions. This is one of the more colorful areas of the City and ranks as top priority in any attempt to control wet weather overflows. Thus Stage 1 consists of constructing 3 shoreline retention basins, one at the Marina Green area at the foot of Pierce Street; the second at the termination of Beach Street on the Embarcadero and the third at the termination of Jackson Street along the Embarcadero.

Each of these will range in size from 200,000 cubic feet up to 1 million cubic feet depending on the alternate selected. The dimensions will be roughly 120 ft. x 60 ft. x 30 ft. deep ranging up to 300 ft. x 120 ft. x 30 ft. deep. The construction will be generally underground with little or no evidence at the surface of the existence of the facility except for minimal required service structures. In all cases multi-use of these surfaces will be developed as intensely as possible.

In addition to these shoreline retention basins, upstream retention basins are proposed at Lombard and Franklin, Columbus and Union, and Pacific and Stockton Streets under Alternate A with the addition of upstream retention basins at Baker and Union, Steiner and Green Streets in Alternates B, C, and D. The flows stored in these retention basins will be routed to the existing North Point treatment plant for treatment. At the completion of this stage the number of overflows per year will be reduced from the present average of 82 to approximately 20, a 75% improvement, although the annual quantity overflowed improves only 40%. Further reduction in overflows will occur upon completion of the necessary increased treatment facilities in the later stages.

STAGE 2. Lake Merced Wet Weather Outfall. The area of the southwest section of the City is a favorable disposal point for treated effluent. Therefore, the construction of a wet weather outfall here provides the alternative of including a dry weather outfall for the disposal of treated effluent from both the North Point and Southeast Plants, if this is required at an early date. If the option of disposal of both dry weather effluent is elected, the outfall line would be designed as a two-compartment conduit with the wet weather discharges, approximately 10,000 feet long and in 60 foot depth of water, through diffuser ports that would result in a surface field during wet weather. This provides the best protection for gravid crab and other benthos organisms for the intermittent discharges which occur during wet weather. Dry weather discharges would be through an outfall 25,000' long and in 90' depth of water, resulting in a submerged field, since this will provide the dilution for the continuous discharge of dry weather treated effluent, and also meet the esthetic requirement.

STAGE 3. Ocean Beach Transport System. The construction of a tunnel between Fulton Street and the Lake Merced combined outfall area provides the first link in ultimately transferring wet weather flows to the Lake Merced Treatment Plant site.

This will also be the primary transfer conduit for all of the retention facilities in the Sunset and Parkside areas, and is a sewer approximately 4½ feet in diameter as it traverses Golden Gate Park, and when emerging from the southerly border of the Park it becomes a tunnel section through the western part of the Sunset district moving southward to link up with the outfall at Lake Merced. This conduit is a prerequisite to the construction proposed in Stages 4 and 5.

STAGE 4. Richmond District/Lobos Creek Retention Basins. A shoreline retention facility is proposed in the Sea Cliff/ Baker's Beach area which will consolidate the existing four outfalls in that vicinity into one overflow outfall. Moving inland easterly of this location, a series of 5 retention basins are proposed along Lake Street, one at California and 28th Avenue, and another at Geary Street and 23rd Avenue, to provide upstream inland retention. The basins will be located at street intersections and will be generally 50 to 60 feet wide except for the one at 14th Avenue which will be 120 feet wide, the lengths would be approximately 300 feet generally except at Lake and 8th Avenue which will be approximately 600 feet and at California and 28th Avenue which will be approximately 140 feet. The depths of these basins range between 15 and 40 feet. These sizes are all maximum sizes which would result if Alternate D is selected. They will range downward to approximately 1/6 of the required volumes if Alternate A is selected. The differences would generally be in terms of length rather than in the other dimensions.

Upon the completion of this phase, the number of overflows will be reduced to approximately 20 per year for this area, until Stage 5 is completed, when they will be reduced further.

STAGE 5. Lake Merced Water Pollution Control Plant. This is the first phase of construction of the Lake Merced Plant, and it is proposed that the initial capacity be 325 MGD. This capacity would provide the equivalent of a tenth of an inch per hour treatment rate for the whole of Richmond-Sunset sector of the City. Upon completion of this facility, the potential of reducing overflows to 8 per year in the case of Alternate A, up to once in 5 years in the case of Alternate D would exist for the western side of the City. At this point, the option also exists to make the Lake Merced facility a combined wet and dry weather plant, with the advantage that it would then be permanently staffed all year round, and alleviate the obvious operational problems associated with intermittent wet weather treatment. It also allows the phasing out of the Richmond-Sunset treatment plant, thereby releasing to park use the area now occupied by that plant.

STAGE 6. Richmond-Sunset Retention Facilities. In this phase, three shoreline retention basins, at Vicente Avenue, Lincoln Way and Fulton Street, and eight through fifteen inland retention basins, depending on the Alternate selected, will be constructed. The topography in this area slopes generally from the central hill mass to the ocean, and is ideally suited for the construction of inland retention structures; the sizes of these structures, at the maximums in Alternate D, range in length from a little over 100 feet to 400 feet, depending on location, in depth from 25' to approximately 35', and in width from 50 feet to 200 feet. Sizes for Alternates A, B and C would be proportionately smaller. They are generally a minimum of four feet below the surface, and exclusively in the street area. The basins are placed so as to relieve the sewers downstream from being overloaded during heavy downpours, and flooding should be relieved considerably in this area at the completion of this plan. The multi-use possibilities for these facilities are limited, because in every case, they are located in street areas on the alignment of the existing sewer systems.

STAGE 7. Lake Merced Tunnel and Retention Basins. The Lake Merced Storage Tunnel serves the dual function of providing upstream storage capability for this area, and as the final leg of the central storage and transport tunnel to be completed in stages 8, 11 and 14. Beginning below the vicinity of Junipero Serra and Lyndhurst, proceeding westerly to the termination at the site of the proposed Lake Merced treatment facility, the tunnel consists of 34 foot diameter sections, approximately 800 ft. long in Alternate A, ranging upward to 4700 feet long in Alternate D, for the storage segment, and a total of about 11,000 ft. of 16.5 feet diameter transport tunnel, integral with the storage segments where they occur. The tunnel will have a minimum of four feet of cover in the Lake Merced vicinity increasing to approximately 100 feet in the inland areas.

In this stage, two retention basins are also proposed, one at Brotherhood Way and Thomas More Way approximately 100' wide by 200' long by 30' deep under Alternate D, ranging downward to one-sixth the volume in Alternate A. A second retention basin is proposed in the vicinity of the existing Lake Merced Pumping Station on John Muir Drive.

With the completion of stage 7, the Northern and Western sections of San Francisco will be brought up to the wet weather control levels projected in the four Alternates.

STAGE 8. Candlestick/South Bayshore Retention Facilities. In the future plans for this area an intensification of recreational and residential development is projected. Therefore, this area is of first priority for wet weather control after the completion of the Western side. A shoreline retention basin is planned for Sunnydale Avenue, which under Alternate D will be approximately 120 ft. wide x 500 ft. long x 35 ft. deep, and proportionately smaller for Alternates A, B and C. A second retention basin is proposed at Yosemite in the South Basin area. There are presently 3 outfalls in this vicinity, which will be consolidated with this retention basin. This basin will be approximately 200 ft. wide x 500 ft. long x 30 ft. deep under Alternate D ranging down to approximately 1/6 of this size for Alternate A. The location of this basin in a planned light industrial area lends itself to a multi-use development of this surface area for structures for an industrial park. Another possible use is an incorporation of park areas on the surface to allow viewing and recreation in this area. Another inland retention basin is proposed at the vicinity of Sommerset and Wayland Streets which will be approximately 600 ft. long x 40 ft. wide x 35 feet deep under Alternate D and range down to 100 feet long x 40 feet wide x 36 feet deep under Alternate A. Typical of inland retention basins, this will be constructed in the street areas, presenting minimum evidence of its existence at the street surface. At the completion of stage 8, the overflow frequency in the Candlestick/South Bayshore area will be reduced from 82 to approximately 20.

<u>STAGE 9</u>. <u>Central Storage and Transport Tunnel, Southeast District</u>. In Stage 9, another link in the proposed central tunnel is planned for construction. This section of the tunnel will consist of 34 foot diameter segments, totalling approximately 1800 feet in length, and approximately 1700 ft. of 16 foot diameter transport tunnels under Alternate A; under Alternate D, the 34 ft. diameter storage tunnel length will be approximately 11,000 feet long. Again, this section of the tunnel has the dual purpose of storing runoff for this area, and conveying flows to the Lake Merced Water Pollution Control Plant. It will connect with the tunnel constructed in Stage 7 at Junipero Serra Boulevard and Lyndhurst.

<u>STAGE 10</u>. <u>Expansion of Lake Merced Water Pollution Control Plant to 550 MGD Capacity</u>. The expansion of the Lake Merced

facility at this stage will result in the reduction of overflows to the levels projected in the various Alternates for the Candlestick/South Bayshore area.

STAGE 11. The Central Storage and Transport Tunnel - Central District. The third segment of the central tunnel will be constructed in this phase, consisting of approximately 600 ft. of 34 ft. diameter storage tunnel and approximately 840 ft. of 10-1/2 to 11-1/2 ft. diameter transport conduit, under Alternate A. Under Alternate D, the 35 ft. diameter storage tunnel will be increased in length to approximately 5700 ft. It will extend from Duboce and Fillmore Streets, where there will be approximately 40 ft. of cover, to the vicinity of 27th Street and Sanchez where there will be approximately 5 ft. of cover, where it will connect to the tunnel constructed in Stage 9. The completion of this phase will, to a great extent, capture the upstream runoff from the area draining to China Basin.

STAGE 12. Islais Creek Shoreline Retention Basin and Related Sewers. A shoreline retention basin will be constructed in this stage in the vicinity of the head of Islais Creek. It will be approximately 300 ft. wide by 600 ft. long with an average depth of 35 ft.; the top of the retention basin will be at the surface, and thus there could be dual use of this surface for Port activities, or for development of park and recreational facilities. At present there are six outfalls draining into or near Islais Creek which will be consolidated through interceptors into this retention basin. The Port of San Francisco plans extensive LASH facilities in the Creek, and careful planning and close coordination between the Port and the Department of Public Works, with the participation of City Planning, should result in a well integrated development for this area. The viewing of maritime activity is one possible use that can be accommodated from the surface of this basin.

A smaller basin is planned for the India Basin area which will consolidate the existing three outfalls. The basin will will be approximately 200 ft. long, by 40 ft. wide by 20' deep under Alternate D, scaling down to 125' x 40' x 20' for Alternates A, B and C. This facility can be integrated into the planned recreational area, and small boat harbor and marina, providing pedestrian access to the shoreline.

Included in this stage will be the construction of a force main to deliver flows up to the Central Tunnel for conveyance to the Lake Merced facility.

STAGE 13. China Basin Retention Facilities. At the head of the Channel which terminates in China Basin is the outfall for the Division Street sewer, a major sewer system; a shoreline retention facility is necessary to control wet weather overflows at this point. This facility will be approximately 180 ft. wide x 900 ft. long x 35 ft. deep under Alternate D, reduced in length to approximately 200 ft. under Alternate A. As in the case of the Islais Creek retention basin, multi-use possibilities exist in terms of integration with planned small boat docking facilities, park and viewing areas, and the re-opening of 6th Street across the channel. Seven existing outfalls presently discharging into the channel will be consolidated and routed to the retention basin, which will have a single overflow outlet, overflowing from 8 times a year maximum to once in five years, depending on the Alternate selected.

Another shoreline retention facility is planned in the Central Basin area at the end of Mariposa Street. This will be a small facility approximately 60 ft. wide x 280 ft. long at the maximum, and 30 ft. deep. The surface of this basin has the potential of being developed into a park and recreational area for viewing activities in the central basin area and beyond into the Bay. Close coordination with City Planning and the Port will result in an attractive solution for this development. An upstream retention basin will be constructed at Valencia Street and 20th Streets, approximately 40 ft. wide by 340 ft. long by 27 ft. deep under Alternate D, and proportionately shorter for the other alternates. This retention facility will be entirely in the street area on Valencia.

STAGE 14. Central Storage and Transport Tunnel, North Section. The construction of this tunnel will complete the storage and transport central tunnel which provides the capability of transferring all flows from Bay discharge to ocean discharge. It will consist of approximately 500 ft. of 34 ft. diameter tunnel under Alternate A and scaling upwards to approximately 3,000 ft. of 34 ft. diameter tunnel under Alternate D. 7,200 feet of 9-1/2 ft. diameter transport conduit will also be constructed in this stage. The top of the tunnel will be generally 5 ft. to 20 ft. below the surface, and the tunnel will begin at the section of Larkin and Eddy Streets and end at the intersection of Duboce and Fillmore Streets. This stage completes all of the storage facilities construction for the Master Plan, and provides all the required transport facilities to the Lake Merced Water Pollution Control Plant for treatment and discharge into the ocean, except for the delivery conduit from the North Point Water Pollution Control Plant.

STAGES 15 AND 16. Expansion of Lake Merced Water Pollution Control Plant. With the completion of Stage 14, the next appropriate step is the expansion of the Lake Merced Plant to 820 MGD, providing the capability of treating all flows presently being handled by the Richmond-Sunset and the Southeast Water Pollution Control Plants. The completion of this phase, in combination with the hydraulic capacity at the North Point Water Pollution Control Plant, will result in the control levels projected under the various Alternates. If conditions are such that the North Point Water Pollution Control Plant must be phased out then an additional Stage 16 is proposed to transfer the treatment capacity from North Point to Lake Merced making the latter a 1000 MGD facility.

CHAPTER VIII

CONTINUING STUDIES

The studies leading to the Master Plan presented in this report, in most part, were designed primarily to establish feasibilities of the concepts embodied in the report, and were not carried to the degree necessary for detailed engineering design of the plan elements. The complexity of the rain-fall runoff process, the scarcity of base line data from which to measure process effectiveness, particularly in the areas of constituent levels in the receiving waters, characteristics of watershed runoff, and dynamics of pollutant buildup and flushing in catch basins and sewers, were all studied on a gross scale by use of typical samples. In all cases, conclusions reached were conservative, and were arrived at only when a particular study had progressed to where the direction was firmly established. However, it is a basic recommendation that five years of hydrologic data acquisition and analysis be continued prior to the detailed design of the bulk of the Master Plan elements. This need not delay the design of certain of the phases, such as the Lake Merced WPCP, outfall, and some upstream storage facilities. Specific areas of further study and evaluation are described in the following sections.

Rainfall-Runoff Studies

The assumption was made in the derivation of all runoff volumes, treatment plant capacity requirements, and projected overflow volumes, that the runoff percentage would be approximately 65%. A review of previous work in San Francisco shows that in the residential neighborhoods, the percentage of area covered by structures or paving ranges between 65 and 90 per cent, depending upon the density of development in a particular area. For this range of imperviousness and times of concentration in the 15-20 minute range, a runoff coefficient of 65% is reasonable. In the more densely developed downtown areas, however, the runoff coefficient would be higher, probably in the 70%-75% range, whereas, in the areas of San Francisco with more open space, the coefficient would probably be in the 55%-60% range. The following table shows the approximate composition of land use in San Francisco:

Classification	Percentage of Total Area
Residential	30%
Streets	25%
Public *	23%
Vacant	8%
Commercial	5%
Industrial	5%
Utility	3%
Institutional	1%

* Approximately half for recreational

Applying appropriate runoff coefficients to each of these, results in a computed coefficient of approximately 65% as a citywide average.

The basis for most data on runoff coefficients has been directed to the calculation of peak runoff from rainfall intensity. In calculating runoff volume, however, the coefficient of runoff is a variable function of intensity of rainfall and the physical characteristics of the watersheds, each of which varies with time. To establish true coefficients, sewer flow hydrographs and related rainfall occurrence must be measured over a long enough period to establish consistent correlations. Reliability of such measurements is proportional to the duration of the data acquisition period, and when the total Master Plan program cost is considered, the accurate determination of these relationships is essential.

Concurrent with the above study, rainfall patterns and directions need to be studied further. Preliminary results from the S.F. Hydrologic and Hydraulic Data Acquisition system show that the only portions of the City are subjected to the higher rainfall rates at any given time, as detailed in Chapter V. Before any conclusions can be reached, however, more significant data must be obtained.

Data Acquisition Program

The need for data in all phases of this program has been evident through all of the studies. Therefore, efforts have been initiated to measure rainfall and runoff quantitatively with the San Francisco Hydrologic and Hydraulic Data Acquisition and Recording System, which is block-diagrammed in Plate VIII-1.

At the writing of this report, the system installation is one of the most advanced in the world and is approximately 80% complete. Some data has been recorded during a portion of the 1970/1971 rainfall season. Major difficulties have arisen in the remote monitoring devices and in communication from them to the central recording station, as well as in the software programs. At present, calibration of the 30 raingages, and the installation of the eventual 120 flow level monitors is proceeding. The 1971-1972 rainfall season will provide meaningful data on the rainfall-runoff process provided that these difficulties can be resolved. The feasibility of providing city-owned communication circuits as a part of the control system will have to be considered if the dependability of telephone circuits cannot be improved.

Pilot Retention Basin

The development of firm operating procedures for storage basins should be preceded with the construction of an inland retention basin as a full scale model study. This would serve several purposes:

1) Establish a precedent from which to refine the cost estimates for future retention facilities;

2) Check the effectiveness of the separation structure in diversion of floatables and solids to bypass the storage compartments;

3) Determine the maintenance requirements for the retention facility itself;

4) Provide the basis for operational rule tables for the facility under varying rainfall occurrence, and selected treatment rates.

Oceanographic Monitoring

In order to assess the impact of treated effluent discharge to the Gulf of the Farallones, the oceanographic study already in progress should be expanded to include the establishing of base line parameters. These base line parameters will include complete physical, chemical and biological surveys at the selected disposal locations in the Gulf of the Farallones. Included among the physical parameters to be investigated are such items as bottom topography, bottom materials identification, sediment mobility, floatables, temperature, color, turbidity and any additional physical parameters deemed necessary. The biological sampling will include planktonic organisms, benthic organisms and several fishes. The chemical parameters to be investigated will include COD, dissolved oxygen, pesticides, grease, nutrients, pH and possibly others. A heavy metal analysis will also be conducted as a part of the sampling program.

This sampling program is to be conducted before the Lake Merced plant or the outfall is in operation to provide a baseline so that future monitoring can be measured against this base. The future monitoring will enable the City to evaluate any changes at the disposal site, if they occur, and consider correctional activities, if necessary.

Treatment Plant Studies

The treatment plant studies are concerned with the development of the Lake Merced WPCP. A conceptual flow diagram has been developed which appears feasible but which has not been pursued to solve all the problems currently envisioned. Among the problems to be solved are the transportation of the Lake Merced sludge to the Southeast plant for treatment. The foremost part of the study problem is the increased amount of sludge which will be produced at Lake Merced during wet weather, which is in the order of three times the daily sludge now handled under existing dry weather conditions. The method employed to handle these increased solids loadings without the creation of nuisance conditions must be evaluated beyond the level which was completed for this report.

The first stage in the development of the Lake Merced plant includes a wastewater reclamation demonstration facility which will be used to verify and quantify the unit processes being considered for reclamation treatment.

Additional pilot plant work will be conducted prior to the construction of the Lake Merced plant. This work will be utilized to optimize the unit processes developing specific design criteria such as overflow rate, detention times and settling tank velocity using the proposed Lake Merced influent streams under the stages of development considered.

Real Time Control

A real time control simulation capability must be developed to determine operational feasibility of central computer control facilities. Data must be obtained from the prototype to develop system response tables historically. These tables will be used for real time control.

Diagrammed in Plates VIII-2, 3, and 4 are the major tasks and operations of the recommended program to achieve real time control. The development phase has already begun with the accumulation of data for dry weather flow tables, and some rainfall data. The results of this phase will be a preliminary set of system response characteristics which will be statistically correlated to establish zones of confidence levels. During the course of this development phase, a prediction phase will begin in which the preliminary results of the development phase will be used for prediction and then cross-checked against measured data for confirmation. The results of this confirmation will again be statistically examined to establish confidence levels. Concurrently, the development phase will continually be updated to strengthen the confidence levels and to establish repeatability of system responses to rainfall input.

The objective of the first two phases is the development of predictability based on historical data, and to determine the appropriate design probability level at which control decisions should be made.

Phase III, the real time phase shows the control options that can be exercised to effectively manage the system. The pilot retention basin previously mentioned would serve to verify the predicted results for the various control options and to modify, if necessary, the conclusions reached in the first two phases.

Implementation

The implementation of continuing studies presents several alternative methods to develop software for a dependable real-time control system:

1) The use of existing engineering staff to perform the analysis of the collected data, and the formulation of operational rule tables for the control device.

2) The use of the Electronic Data Processing staff in performing the detailed analysis, and formulation of the rule tables, under the direction of the engineering staff, and ultimately, the integration of all computer activity into their existing system.

3) Contracting to consultants the analysis and formulation of rule tables, with their recommendations on the best approach of how to implement the conclusions.

4) The development of computer-oriented capability and equipment within the Department of Public Works.

The first alternative has the advantage of the involvement of personnel most intimately involved with all aspects of the problem. The disadvantage is that the diversion of engineering personnel to programming activity is expensive, and the expertise required to properly use all of the available computer technology is limited. However, this alternative must be a part of the final recommendation.

The second alternative possesses the advantage that some expertise exists in programming, and a large computer is available. The disadvantage is that the primary activity of this group is oriented toward accounting, and the operation is geared in this direction. Thus, if the data acquisition and control activity is integrated into the system, it must be done on a shared time basis, and the obvious undesirability makes this an unattractive alternative.

The third alternative is probably the most expensive approach to the solution. The advantage is that the specialized capability required is available with selected consultants; but the disadvantage here is that a certain amount of initiation time is required to reach the required level of activity and involvement, and the long-term nature of the work requires the contracting of consultants over extended periods of time. The management of this effort would also consume large quantities of staff time.

The fourth alternative is, perhaps, the most practical approach to the solution of the problem. It possesses the best elements of the first three alternatives, and assures the continuity of effort which is required. Thus it is recommended that a group of computer-oriented personnel be assembled to perform the necessary pre-operation programming effort required, that they be under the direction of the engineering staff, and that all of the required computer activity be coordinated by this group. Upon development of operational software components of the central control unit by this staff, the efforts of this group will then be dedicated to implementing real time operational control. EDP equipment will be utilized for production processing phases of this effort.

Summary:

The following recommendations are made for a reasonable continuing data acquisition and development of process programming considered necessary to development of the alternate Master Plans presented in this report.

1. A minimum of five years should be devoted to data acquisition and analysis in order to establish detailed design bases and operational rule tables.

2. The data gathering work program must address the quality of constituents including those incident on the watersheds and sewers, and also base line data in the receiving waters particularly in the present and prospective ocean discharge sites.

3. The staff of the Department of Public Works should be increased by the addition of four computer systems and programming personnel with basic knowledge of sanitary and hydraulic engineering parameters so work can be initiated to develop the experience table necessary for real time control for effective management of the system including construction of a pilot inland retention basin.

The annual cost of the above five-year data acquisition and development program is estimated at $600,000, an investment which is considered necessary to achieve the predicted control effectiveness, particularly of the least expensive Master Plan alternatives.

CHAPTER IX

FINANCING PROGRAM

The final portion of this Master Plan Report involves the financial planning necessary for the funding of construction and maintenance and operation of the facilities previously described. This financial planning satisfies in part the Regional Board's present requirements which stipulate that the City Master Plan include a program for financing the construction of the Master Plan and the setting of dates necessary for the bond elections.

This report recommends that the City submit a cash flow program of possible bond issues for alternative overflow occurrence control schemes which will reflect the City's portion of the costs allocated for the total waste water management program. A firm bond issue schedule can then be established after these variables are determined:

A) The degree of wet weather control, that is the average number of overflows annually (Alternates A to D),

B) The level of dry weather treatment necessary to meet long range requirements and policy.

C) The time period for completion of the various program portions.

D) Federal and State grant programs and available assistance.

With the preceding unknown factors, the financial planning presented in this chapter must be sufficiently flexible to cover a matrix of cost alternatives. To facilitate a comparison of financial plans, the following assumptions were made:

1) Interest rate = 6%
2) The present worth of each scheme will be based upon 1974 project costs.
3) The capital cost expenditures of a Master Plan for the project life will be in uniform annual increments.

Capital Costs for Treatment and Control

The cost for the wet weather treatment vs. accomplishments has been tabulated in Plate VI-15. A brief summation of the wet weather alternatives follows:

Alternative	No. of Overflows Per Year	Cost Millions of 1974 Dollars
A	8	$333
B	4	396
C	1	522
D	0.2	665

The dry weather cost summation which follows includes in Scheme I the minimum first cost for dry weather treatment applicable to a combined dry weather and wet weather solution and, in Schemes II and III, the increased cost effects of implementing the dry weather solution separately from the wet weather solution based upon either differences in time schedules or on funding. Plates VI-12 and VI-13 provide the basis for the following cost summation.

DRY WEATHER TREATMENT

(Cost in Millions of 1974 Dollars)

	Level I	Level 2	Level 3
Scheme I - Most Economical Method	$42	$83	$130
Scheme II - R-S & SE to Ocean	73	125	172
Scheme III - R-S, SE and NP to Ocean	84	136	183

The total wet weather and dry weather costs for the possible 36 combinations are shown on Plate VI-14.

Capital Costs, Transport System

As noted in Chapter 8, the total cost of replacing all the sewers in the City which are inadequate to carry the design storm flows has been estimated to be $150 million. Implementation of upstream storage basins in any of the alternative wet weather plans will reduce the inadequate sewers by one-third and will reduce the costs for replacement to $100 million dollars. It is assumed that the replacement of inadequate sewers will be accomplished in a thirty-year program.

A further capital cost is the cost of replacing sewers which have deteriorated to the point of failure either by virtue of age, chemical attack, or changes in the necessary soil support conditions.

This sewer replacement cost was estimated based upon the following:

1. 750 miles of sewers in the system are less than 30 inches in diameter. About 120 miles of these small pipe sewers will require replacement over the next forty years.
2. There are about 75 miles of brick sewers in the City system which will be replaced over the next forty years.

This portion of the replacement program will require an annual expenditure of $3.75 million.

The above four capital cost areas represent the total capital expenditures anticipated to be required for a comprehensive water pollution control program. An additional set of costs affecting the cash flow program are the annual maintenance and operation costs.

Maintenance and Operation - During Master Plan Construction

The cost associated with maintenance and operation will depend upon the level of treatment provided for dry weather and will fluctuate as the various portions of the Master Plan project are completed.

The dry weather maintenance and operation costs will vary from $4.7 million dollars (1974) for first level treatment at the individual plants up to 7.5 million dollars (1974) for second level dry weather treatment at the individual plants. These costs may be reduced to 7.2 million dollars (1974) by consolidation of facilities. The additional costs of wet weather treatment to first level effluent will range from about 0.1 million dollars (1974) for a single wet and dry weather plant to 1.1 million dollars for a separate wet weather facilitage. If second level dry weather treatment is implemented and the plants are consolidated then, the maintenance and operation cost are 7.0 million dollars (1974) during a twenty to thirty year wet weather construction period. This assumes a dry weather program of about five year's length.

Maintenance and Operation - After Master Plan Construction

Upon completion of the Master Plan construction phase the Maintenance and Operation (M&O) costs will have risen due to increased facilities and treatment. Some costs will have been reduced by consolidation. Estimates were made based upon past expenditures, and future treatment levels, assuming optimum consolidation and include minor sewer construction costs which occur every year.

The total annual cost is divided into the two previously mentioned categories, minor sewer construction additions and maintenance and operation. The two categories include the following:

Minor Sewer Construction	Maintenance and Operations
1. Side Sewers	1. Dry and Wet Weather Treatment Plant
2. Culverts and Catchbasins	2. Culvert and Catchbasin Cleaning
	3. Sewers
	4. Outfall(s)

The annual cost of the sewer construction items are estimated to be as follows:

Annual Minor Sewer Construction Cost

Side Sewers	Culverts and Catchbasins
$220,000	$70,000

It must be noted that the above costs of construction of side sewers are borne by the property owner and further essentially 100% of the cost of the culverts and catchbasins are paid for by State gas tax funds.

The Maintenance and Operations costs are expected to be as follows:

Annual Maintenance and Operation Cost

Dry & Wet Weather Treatment		$7,300,000
Culverts and Catchbasins		280,000
Sewers		1,500,000
Outfall		90,000
	Total	$9,170,000

The costs for maintenance and operation of the dry weather and wet weather treatment are optimum cost figures based upon the wet weather treatment alternatives and the dry weather treatment levels. The M&O costs for dry weather treatment levels ranged in cost from $7.2 million to $7.5 million (1974).

The wet weather M&O cost ranged from approximately $0.1 million (1974) for the alternatives estimated.

The cost of the upkeep of culverts and catchbasins was obtained by extrapolating data from the City's Annual Reports for the fiscal years 1964 to 1969 inclusive.

The cost of maintaining the sewers during dry and wet weather periods was obtained by extrapolating to 1974 data from Annual Reports for the fiscal years 1965 to 1969, inclusive. The outfall M&O cost was forecasted by a consultant as being 0.3% of the initial construction cost.

Funding

The funding sources available to the City are local, State and Federal.

Local

Local funding is available from *ad valorem* charges, user charges, and bonds. The *ad valorem* charge is a property tax charge. User charges now include the charges to industry under the 1971 Industrial Waste Ordinance and a sewer service charge.

The 1971 Industrial Waste Ordinance established a specific fund into which all revenues collected from the enforcement of the 1971 Industrial Waste Ordinance are to be deposited. It is expected that after the industrial waste program is under full operation, the revenue collected will be approximately $500,000 annually. This amount approximates the estimated cost of administering this program and the additional cost of treatment caused by the industrial waste discharge.

In 1971 the City also adopted a sewer service charge based on water consumption to finance the maintenance and operation of all sewerage facilities and to pay for the interest and redemption of sewerage and water pollution bond issues authorized through the year 1970. It is expected that the present sewer service charge will generate about $13 million (1974 dollars) per year. At this time bond funds for water pollution control facilities are available in the amounts of $65 million dollars approved in the 1971 bond issue for dry weather and wet weather facilities and about $6 million from previous bond issues for the sewer replacement and flooding control programs. It is obvious that these funds alone will not be sufficient to meet the capital needs of the water pollution control program beyond the next few years. Other funds in the form of further bond issues and aid from state or Federal programs will be necessary.

Federal and State Grants

With the passage of the 1970 $250 million California State Clean Water bond issue, the first State grants were made available for treatment facilities. This resulted in an increase in Federal allocation from 33% to 55% provided that the State contributes 25%. This would require a discharger to finance the remaining 20% of the total project cost. However, the State Water Resources Control Board, in administering the Federal funding program, is not obliged to certify the total Federal 55% which would result in less than the 80% maximum total grant. Further, wet weather projects are considered, at this time, a lower priority than dry weather projects, and are not scheduled for either State or Federal grants.

There is also limited Federal funding available under the Housing and Urban Development Act (HUD) for construction of sewer facilities, but the City's eligibility is dependent on the acceptacne by HUD for treatment of wet weather flow in lieu of sewer system separation.

Maximum Percentage Funding Contributions

The previous categorizing of various items in the Master Plan is to reference those various categories to possible funding sources, namely; local, State, and Federal. These sources with their maximum possible percent contribution are shown on Plates IX-1 and IX-2.

The total percentages for any one item cannot exceed 100% and the City would seek to spread the cost burden to include a maximum of State and Federal funds, but it must be recognized that there exists the possibility that the City would have to pay the total cost.

It can be noted from Plates IX-1 and IX-2 that for various portions of the total program the City may pay from 0 to 100 percent of the costs. However, for treatment facilities financial assistance up to 80% of the first cost is possible. Under present provisions Federal or State assistance with the annual costs is not possible. Thus the City must carry the full burden of the increased maintenance and operations costs.

Bonded Indebtedness Limitations

With regard to local funding of facilities two basic possibilities exist; "pay-as-you-go" financing yearly either from City revenues or from the sewer service charge, or, general obligation bonds providing large amounts of capital funds and which may be amortized over a period of years by City revenues or the sewer service charge. The approach used in this analysis is that general obligation bonds will be used as the most practical approach to financing the large expenditures contemplated.

Under provision of the Charter of the City, the dollar amount of bonds sold and outstanding at any time may not exceed an amount equal to 12% of the City's assessed property valuation. Exceptions to this limit are provided for bonds paid for by revenue such as Airport and Water Department bonds, certain school bonds and bonds to be amortized by the sewer service charge. Plate IX-3 tabulates the financial status of the City in this regard for the period from the fiscal year 1955-56 through 1970-71.

The limit of future bond sales is significant in two regards. First, the closer to the limit the City comes in outstanding bonds, the higher the interest rates for new bond sale will be. The second and more significant point is that by law an upper limit is established for future bond sales. Obviously, it is not desirable to utilize the full bonding capacity of the City for both reasons of economics and of City financial flexibility for future needs.

It is also worthy of noting that the unissued bonds subject to the indebtedness limitation are nearly equal to the future bond sales limit and have exceeded the limit in certain years. This depicts the near future needs of the City indirectly.

In developing a financing program for this Master Plan certain assumptions as to fund obligations are required in order to allow the development of a reasonable program with the least number of variables. Based upon the analysis of the annual expenditures for maintenance and operation and the present amount of authorized but unissued bonds, it is assumed that the sewer service charge will be adjusted to provide sufficient funds to pay the maintenance and operation costs and to amortize the presently authorized water pollution control bonds. This means that only the bonds that will be needed in addition to those existing will be subject to the 12% bonded indebtedness limitation.

Future Bond Issues Required

The remaining factor required to complete the development of a financial program is to determine the necessary additional bond funds for the facilities described in this report and to develop a tentative schedule of bond issues for the program. There are, however, still several variables to be considered. These are:

a) Length of program,

b) Dry weather alternative,

c) Wet weather alternative, and

d) State and Federal grant percentages.

Obviously there are limitless combinations that might be considered for program purposes. However, it is not necessary to consider all possibilities directly. Rather some reasonable assumptions can be made and a program developed which can be modified if future circumstances dictate changes in assumptions.

Length of Program

Two time periods are to be considered in this report, 20 years and 30 years. Any total program shorter than 20 years is considered to be too short to reasonably design, finance, and complete construction for a program of this magnitude. The thirty year program is likewise considered to be the longest period for which a plan of this nature should be considered.

Dry Weather Alternative

Consideration of all present policy of the State and Federal regulatory agencies indicates that for purposes of planning the second level of treatment will be adequate within the time periods to be considered. It is also doubtful over the long term that the discharge of this level of effluent will be allowed into an estuary of such large value as San

Francisco Bay. A balanced water pollution control program would also indicate that, prior to the institution of the very high degrees of dry weather treatment, wet weather control be attained. This report assumes that alternate III-2 (page IX-3) which provides for near future improvements to the existing plants and transport and discharge to the ocean with ultimate consolidation with the wet weather treatment plant will be required to satisfy the present Federal and State requirements. It must be noted that a potential saving of $50 million dollars could be achieved through implementation of Scheme 1-2 which provides coordinated dry weather and wet weather improvements with no duplication of facilities. These savings are not attainable due to present Federal and State funding priorities for dry weather facilities.

Wet Weather Alternative

No selection has been made regarding the four wet weather alternatives presented. Four alternative programs will be presented.

State and Federal Grants

Of all the variables presented, this one is the most difficult to analyze. Grants may range from zero to eighty percent depending upon the availability of funds and the acceptability of the projects to rhe regulatory agencies.

For the purposes of this report grants for treatment facilities have been assumed to be eighty percent. This will allow the development of a program of City financing. If lower percentages are forthcoming, then it is the intent of this report that the yearly financial commitment of the City would remain the same with the extension of the length of the program as necessary to complete the facilities within the annual cost commitment.

Program

Given the above conditions, then the City's financial commitment program is depicted on Plates IX-4 through IX-9. Plate IX-4 tabulates the net costs to the City for the various portions of the waste water management program under the above conditions. Plate IX-5 summarizes the total bond issues required for the 20 and 30 year programs together with the interest costs for the bond issues.

Based upon the length of the programs (20 years and 30 years) and the required additional bond funds tabulated in the previous two plates, a bond issue schedule for the two programs are shown on Plates IX-6 and IX-7 for the twenty year and the thirty year programs. Bond issue dates were selected to meet the needs of the programs over approximately equal time periods resulting in a cash flow as shown. The 1972 date for the sewer bonds will be necessary to provide funds to continue the flooding abatement and sewer replacement program. The 1971 bond issue will provide funds for the dry weather and wet weather treatment and control programs until the second bond issue date.

For purposes of the scheduling of these future bond issues, it was assumed that a 6% escalation will be necessary. Plates IX-8 and IX-9 tabulate the escalated bond issue costs necessary to meet inflationary trends.

It must be emphasized that if State and Federal grants are not available to meet the 80% assistance shown in the plates used to develop this program, then subsequent bond issues will be required and the program lengths extended for completion.

APPENDIX TO CHAPTER III

DETAILS OF REGIONAL WATER QUALITY CONTROL BOARD REQUIREMENTS

As a result of staff investigation, the Regional Water Quality Control Board has determined that, with respect to the North Point and the Southeast Water Pollution Control Plants, the wastes can affect the following beneficial uses of San Francisco Bay and contiguous waters:

1. Industrial cooling water in the vicinity of the Pacific Gas & Electric Company's generating plants at Potrero and Hunters Point.
2. Swimming, wading, pleasure boating, marinas, launching ramps, fishing and shellfishing.
3. Firefighting and industrial washdown.
4. Fish, shellfish and wildlife propagation and sustenance, migratory bird habitat and resting.
5. Navigation channels and port facilities.
6. Esthetic appeal.

Land uses within 1000 feet of the discharges from these plants include port facilities and transportation.

The Richmond-Sunset plant discharges into the Pacific Ocean. With regard to that discharge, the Board recognizes the following beneficial uses of the adjacent ocean and beach area:

1. Aesthetic enjoyment.
2. Fish habitat, migration and propagation.
3. Sport fishing.
4. Wading (at Baker's Beach and Phelan Beach).
5. Swimming (at Phelan Beach).

Land uses near the discharge points are picnicking, sun-bathing, horseback riding, and aesthetic enjoyment.

<u>Intentions of the Board</u>

In accordance with its legal obligations to control water quality within its area, the Regional Water Quality Control Board has stated its intentions, as related to San Francisco's three water pollution control plants, as follows:

1. To protect public health as it may be affected by waste discharges from these plants.
2. To prevent nuisance, as defined by Section 13005 of the California Water Code.
3. To protect the recognized beneficial uses of the receiving water. The Board will consider protecting beds suitable for shellfishing along the shores of San Francisco Bay after having reviewed a report to be submitted by the State Department of Public Health.

Additional stated intentions with regard to the Richmond-Sunset plant are to make or preserve the waters of the Pacific Ocean suitable for the following beneficial uses at all times:

1. At all places along the Ocean shoreline of the City and County of San Francisco:
   (a) Fish propagation and habitat;
   (b) Sport fishing;
   (c) Aesthetic enjoyment.
2. At all beaches along the shoreline, except Federally-owned beaches on the Presidio, so long as a prohibition against the following beneficial uses is enforced, also excepting beaches within 1500 feet of the Mile Rock discharge;
   (a) Swimming and wading, without designating water-contact sports areas within the meaning of Section 7952 of Title 17, California Administrative Code.
3. At Kelley Cove, offshore from the Sutro Heights Park, north of Balboa Street extended and south of Seal Rocks:
   (a) Surf-boarding, without designating a water-contact sports area within the meaning of Section 7952 of Title 17, California Administrative Code.

RWQCB Requirements for Receiving Waters

The RWQCB No. 2 established standards for the receiving waters at the three water pollution control plants as follows:

Richmond-Sunset Plant - Res. No. 67-2, dated Jan. 19, 1967;
North Point Plant - Res. No. 70-17, dated March 26, 1970;
Southeast Plant - Res. No. 69-44, dated Sept. 25, 1969.

For all three of the plants these resolutions provide that the discharges of wastes shall not cause atmospheric odors recognizable as being of waste origin at any place outside the plant.

For the North Point and Southeast Plants, the requirements provide that the discharge of waste shall not cause:

1. Unsightliness, nor damage to any of the protected beneficial uses resulting from:

    (a) Floating, suspended, or deposited macroscopic particulate matter, foam, oil, or grease in waters of the State at any place; floating oil shall be considered present if in sufficient quantity to cause irridescence. (For the Richmond-Sunset plant there is a parallel requirement but it is stated less broadly as follows: "Deposited macroscopic particulate material or foam of waste origin at any place.")

2. Bacterial concentration in waters of the State at any place within one foot of their surface to exceed the limits prescribed in Section 7958, Title 17, California Administrative Code, at any time; when this bacterial concentration is exceeded in the receiving waters for any reason it shall be met instead in the waste at some point in the treatment process and the discharger may do so as an optional alternate; the Board will accept proof of effective effluent disinfection in terms of factors other than bacterial concentrations if the dischargers documents a sound statistical correlation between such factors and bacterial analysis.

3. Waters of the State to exceed the following limit of quality at any place within one foot of the surface at any time:

Dissolved Oxygen	5.0 mg/l minimum
Dissolved Sulfides	0.1 mg/l maximum
pH	7.0 minimum 8.5 maximum

Any one or more substances in concentrations that impair any of the protected beneficial water uses or make aquatic life or wildlife unfit for consumption.

For the Richmond-Sunset Plant the Board's requirements (in addition to those mentioned above) are that sewage discharged from the "Richmond-Sunset sewerage zone" shall not cause any of the following at any time:

1. Turbidity or discoloration in waters of the State at any place more than 200 feet from the Mile Rock outfall.
2. Dissolved sulfide concentrations greater than 0.1 mg/l within one foot below the surface of the Pacific Ocean at any place.
3. At any place more than 100 feet from the Mile Rock Outfall:

Dissolved Oxygen	5.0 mg/l minimum
pH	6.5 minimum 8.5 maximum

4. At any place more than 300 feet from the Mile Rock Outfall: any substance or combination of substances in concentrations deleterious to fish or other aquatic life (limits to be set later).
5. Ocean waters being protected for swimming, wading or surfboarding (see "Intentions of the Board"), to exceed those standards prescribed in Sections 7957 and 7958 of Title 17, California Administrative Code.

In addition, sewage discharged from the Mile Rock outfall shall not cause the receiving waters at the beaches within 1500 feet of the outfall to exceed those standards prescribed in Sections 7957 and 7958 of Title 17, California Administrative Code at any time that the public is not effectively excluded from those beach areas.

RWQCB Requirements for Waste Quality.

In addition to establishing requirements for the quality of the receiving waters as influenced by the discharge of effluent from water pollution control plants, as given above, the RWQCB has also stipulated that the wastes discharged shall meet, at all times, the following limits:

1. In any grab sample:

   Settleable Matter

   The arithmetic average of any six or more samples collected on any day -----------0.5 ml/l/hr maximum

   (In the case of the Richmond-Sunset Plant, this requirement is stated as "any 24-hour composite sample made up of portions collected in proportion to rate of flow at time of collection.")

   Eighty percent of all individual samples collected during maximum daily flow over any 30-day period-------------------------------0.4 ml/l/hr maximum

(This item is stated for the North Point and Southeast plants.)

Any sample--1.0 ml/l/hr maximum

2. In any representative 24-hour composite sample:

   Toxicity: the concentration of the waste itself at any place within one foot of the surface of the receiving waters----------------------------------10% of the 96-hr TL_m concentration of the waste or discharged, maximum

   (Above requirement for North Point and Southeast Plants).

3. Five-day, 20°C BOD -- Whenever the receiving water dissolved oxygen concentration prescribed above is not met, the BOD removal from the waste, as demonstrated by analyses of 24-hour composite samples of influent and effluent, shall be increased sufficiently to maintain said dissolved oxygen concentration, but BOD removal during any 21 or more days is not required to exceed:

Average	90%
Not more than two consecutive daily determinations shall indicate BOD removals less than	80%

(Above requirement for North Point and Southeast Plants.)

WET WEATHER OVERFLOW REQUIREMENTS

RESOLVED BY THIS REGIONAL BOARD

Board Intent

1. Protect public health as it may be affected by this waste discharge.
2. Prevent nuisance, as defined in Section 13050(m) of the California Water Code.
3. Protect the beneficial water uses listed under "Staff Investigation" above.

   In accordance with Section XVII of its Resolution No. 803, this Board has received a report from the Department of Fish and Game dated August 26, 1968, which describes beds suitable for shellfishing that are located along the Bayshore south of Candlestick Point, along the shoreline of Alameda County in Richardson Bay, Marin County.

This Board will consider the matter of protecting these beds for the taking of shellfish for human consumption after it has reviewed a report to be submitted by the State Department of Public Health in accordance with Resolution No. 803.

4. Reconsider the areal and time limits over which these requirements apply after receipt of additional information on the costs and feasibility of providing waste treatment and disposal facilities needed to treat flows from storms of various intensities and on shoreline development plans.
5. Encourage the investigation of deepwater outfalls for all wet weather discharges.
6. Overflows from the sewer system onto City streets are considered to be waste discharges when they enter Bay waters and are subject to the requirements contained herein.
7. That treatment facilities for wet weather flows be constructed and operated so that the initial discharges are treated sufficiently to comply with the requirements contained herein.

Waste Discharge Requirements

1. The treatment or disposal of waste shall not create a nuisance as defined in Section 13050(m) of the California Water Code.

2. The discharge shall not cause:

   a. Floating, suspended, or deposited macroscopic particulate matter or foam in waters of the State at any place;

   b. Bottom deposits or aquatic growths at any place;

   c. Alteration of temperature, turbidity or apparent color beyond present natural background levels in waters of the State at any place;

   d. Visible, floating, suspended or deposited oil or other products of petroleum origin in waters of the State at any place;

   e. Waters of the State to exceed the following limits of quality at any place within one foot of the water surface:

Dissolved oxygen	5.0 mg/1 minimum When natural factors cause lesser concentrations than this discharge shall not cause further reduction in the concentration of dissolved oxygen.
Dissolved sulfide	0.1 mg/1 maximum
Nutrients	To be prescribed at earliest practicable date

Other substances	Any one or more substances in concentrations that impair any of the protected beneficial water uses or make aquatic life or wildlife unfit or unpalatable for consumption.
Bacterial concentrations	In excess of a median value of 240 MPN coliform per 100 ml, as determined in any five consecutive samples collected at any one station, or any single sample to exceed an MPN coliform concentration of 10,000/100 ml at any time. Whenever either of these bacterial values is exceeded in the receiving water for any reason they shall both be met instead in the waste at some point in the treatment process, provided that at least one sample is collected from each initial portion of waste to be discharged through the outfall. The discharger may demonstrate compliance in the waste stream as an optional alternative.

3. Waste as discharged to waters of the State shall meet these quality limits at all times:

   a. In any grab sample:

pH	7.0 minimum 8.5 maximum

   b. In any representative set of samples:

Toxicity: Survival of test fishes in 96-hour bioassays of the waste as discharged

Any determination	70% minimum
Average of any three or more consecutive determinations made during any 21 or more days	90% minimum

c. In any grab sample:

Grease	25 mg/1 maximum
Settleable matter	1.0 ml/1/hr. maximum

This Board considers these two limits to be goals rather than requirements and will consider requirements for settleable matter, grease and/or floatable matter after reviewing additional information on the costs and other information relative to the feasibility of compliance therewith.

ABBREVIATIONS

ADWF - AVERAGE DRY WEATHER FLOW

COD - CHEMICAL OXYGEN DEMAND

BASS - BAY AREA SIMULATION STUDIES

BOD - BIOCHEMICAL OXYGEN DEMAND

DW - DRY WEATHER

DWR - DEPARTMENT OF WATER RESOURCES

EPA - ENVIRONMENTAL PROTECTION AGENCY

FWQA - FEDERAL WATER QUALITY ADMINISTRATION (NOW WATER QUALITY OFFICE - EPA)

GPD - GALLONS PER DAY

HEM - HEXANE EXTRACTABLE MATERIAL

IPH - INCHES PER HOUR

MCF - MILLIONS OF CUBIC FEET

MPN - MOST PROBABLE NUMBER

MGD - MILLION GALLONS DAILY

NP (NPWPCP)-NORTH POINT WATER POLLUTION CONTROL PLANT

OPP - ORTHO PHOSPHATE PHOSPHORUS

PSI - POUNDS PER SQUARE INCH

PWWF - PEAK WET WEATHER FLOW

RCP - REINFORCED CONCRETE PIPE

R-S (RSWPCP) - RICHMOND-SUNSET WATER POLLUTION CONTROL PLANT

RWQCB - REGIONAL WATER QUALITY CONTROL BOARD

SE (SEWPCP) - SOUTHEAST WATER POLLUTION CONTROL PLANT

SWRCB - STATE WATER RESOURCES CONTROL BOARD

TN - TOTAL NITROGEN

TSS - TOTAL SUSPENDED SOLIDS

USACE - U.S. ARMY CORPS OF ENGINEERS

USGS - U.S. GEOLOGICAL SURVEY

USWB - U.S. WEATHER BUREAU

VCP - VITRIFIED CLAY PIPE